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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXV—No. 5

PRESENTED TO
AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852
N. Y. CIVIL ENGINEERING SOCIETY
BY
WILLIAM P. MORSE

May 1909

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 5.

MAY, 1909.

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NEW YORK 1909

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ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

April 21st, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 108 members and 53 guests.

A paper by C. S. Bissell, M. Am. Soc. C. E., entitled "The Maximum Weights of Slow Freight Trains," was presented by the Secretary, who also read a communication on the subject from Mr. L. A. Riley, 2d.

A paper by W. W. Crosby, M. Am. Soc. C. E., entitled "Sampittic Surfacing," was presented by the author and discussed orally by Messrs. A. H. Blanchard, Francis T. Smith, Philip P. Sharples, H. W. Clark, J. W. Howard, and the author. Written communications on the subject from Messrs. L. W. Page and Prevost Hubbard were presented by the Secretary.

W. W. Crosby, M. Am. Soc. C. E., offered the following resolution:

"*Resolved:* That the Board of Direction of the American Society

of Civil Engineers be requested to appoint a Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use."

The Resolution, being duly seconded, was adopted by a vote of more than 25 Corporate Members.

The Secretary announced the following deaths:

LEWIS FREDERICK RICE, elected Member January 2d, 1889; died April 13th, 1909.

ANDREW ROSEWATER, elected Member October 3d, 1883; died April 17th, 1909.

AUGUST MAYER, elected Associate Member June 1st, 1892; died March 25th, 1909.

Adjourned.

May 5th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, 132 members and 31 guests.

The minutes of the meetings of March 17th and April 7th, 1909, were approved as printed in the *Proceedings* for April, 1909.

The Secretary announced that in accordance with the Resolution adopted by the Society at its meeting on April 21st, 1909, the Board of Direction, at its regular meeting, May 4th, 1909, had appointed the following Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use: Messrs. W. W. Crosby, A. W. Dean, H. K. Bishop, and A. H. Blanchard.

A paper by Myron S. Falk, M. Am. Soc. C. E., entitled "A System of Cost Keeping," was presented by the author and discussed orally by Messrs. William M. Black, Robert B. Stanton, T. Kennard Thomson, Willard D. Lockwood, William B. Reed, and Guy W. Culgin.

Written communications on the subject from Messrs. P. L. Reed, W. C. Hammatt, and Emile Low, were read by the Secretary.

A paper by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., entitled "The Design of Elevated Tanks and Stand-Pipes," was presented by the Secretary, who also read communications on the subject from Messrs. Charles B. Burdick, and O. E. Selby. The paper was discussed orally by Messrs. William M. Black, and James M. Porter.

The Secretary announced the election of the following candidates by the Board of Direction on April 6th, 1909:

AS MEMBERS.

JOHN BENNINGTON BERRY, Chicago, Ill.
OSCAR SIDNEY BOWEN, Spokane, Wash.
WILLIAM HOWELL CALDWELL, Tampa, Fla.
JOSEPH KITTREDGE CHOATE, Cooperstown, N. Y.

THOMAS CURTIS CLARKE, New York City.
CHARLES LEO CRANDALL, Galveston, Tex.
HARRY HAWGOOD, Los Angeles, Cal.
THOMAS NOTTINGHAM JACOB, Herrin, Ill.
WILLIAM PRENTISS MORSE, Newton, Mass.
WILLIAM RODNEY PATTERSON, Chicago, Ill.
HENRY OTIS POND, Tenafly, N. J.
WALTER JUSTIN SHERMAN, Toledo, Ohio.
WILLIS ROBINSON TENNEY, Brooklyn, N. Y.
EDWIN DERICKSON VINCENT, Yuma, Ariz.

As Associate Members.

JOHN HOWARD ALEXANDER, St. Louis, Mo.
WALTER CHARNLEY, Rio Grande do Sul, Brazil.
FRANK LEMUEL CLAPP, Cornwall-on-Hudson, N. Y.
JOHN SEBASTIAN CONWAY, Portland, Ore.
BERTRAM EDWIN CORLETT, Seattle, Wash.
CHESTER CAWTHORNE DERBY, Richmond Hill, N. Y.
ARCHIBALD MARVINE GILBERT, Twin Falls, Idaho.
WILLIAM STEWART GREENE, New Brunswick, N. J.
RUSSELL THEODORE HARTMAN, Des Moines, Iowa.
WILLIAM SYLVESTER HAZELTON, Detroit, Mich.
LARS RASMUS JORGENSEN, San Francisco, Cal.
ALFRED BROUGHTON KISSACK, Leavenworth, Kans.
WILLIAM ARTHUR LAFLER, Rome, N. Y.
CLARENCE BOOTH LAMONT, Seattle, Wash.
WALTER FRANKLIN LINEBERGER, Torreón, Coahuila, Mexico.
THOMAS CLAWSON MARSHALL, Middletown, N. Y.
BERTRAND CLIFFORD MARTIN, Watertown, N. Y.
FRANK DANA NASH, Manila, Philippine Islands.
HARRY BRADFORD POND, New York City.
HARRY IVES SHOEMAKER, Phoenix, Hawaii.

As Associates.

JOHN COFFEE HAYS, Visalia, Cal.
FRANK MAYHEW TALBOT, Glen Ridge, N. J.

As Juniors.

CHESTER DANIEL ASHBROOK, Seattle, Wash.
HARRY MONTIFIORE BERGMAN, New York City.
LYMAN EDGAR BISHOP, Denver, Colo.
MYRON CARLOS BURR, Vancouver, B. C., Canada.
STANLEY GARDNER CUTLER, Manila, Philippine Islands.
FRANK JOHN FELGENHAUER, Brooklyn, N. Y.

Robert Maximilian Feustel, Madison, Wis.
Alvero Charles Gregson, New York City.
Charles Burridge Hawley, Cleveland, Ohio.
Cyrus Pierce Howes, St. Louis, Mo.
Andrew Peach Rollins, Eagle Lake, Tex.
Harvey Hillyer Sims, Savannah, Ga.
Thomas Harrison Winchester, Macon, Ga.

The Secretary announced the transfer of the following candidates by the Board of Direction on May 4th, 1909:

From Associate Member to Member.

Philip Asfordby Beatty, Baltimore, Md.
Arthur Horace Blanchard, Providence, R. I.
Clarence Austin Crane, New York City.
Frank Tenney Darrow, Lincoln, Nebr.
Leonard Henry Davis, Sault Ste. Marie, Ont., Canada.
Frank Lawton Getman, Havana, Cuba.
Harry Thomas Paterson, Beaufort, N. C.
Samuel Payson Waldron, East Orange, N. J.

From Junior to Associate Member.

William Burton, Detroit, Mich.
William Hale Phillips, Spokane, Wash.
Walter John Plogsted, New York City.
William Purves Taylor, Philadelphia, Pa.

The Secretary announced the following deaths:

Amzi Lorenzo Barber, elected Fellow March 19th, 1886; died April 18th, 1909.

John Arnold Ubsdell, elected Associate June 7th, 1905; died April 23d, 1909.

Mace Moulton, elected Member June 4th, 1884; died April 27th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

May 4th, 1909. President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Christie, Churchill, Harrison, Horton, Kittredge, Knap, Macdonald, Noble, Pegram, Schneider, Stearns, Stuart, Thompson, and Tillson.

Ballots for Membership were canvassed, and 14 Members, 20 Associate Members, 2 Associates, and 13 Juniors were elected, and 4 Juniors were transferred to the grade of Associate Member.

Eight Associate Members were transferred to the grade of Member.

The following resolution was adopted:*

"*Resolved,* That a Special Committee be appointed to Report on Bituminous Materials for Road Construction, and on Standards for their Test and Use, to consist of Messrs. W. W. Crosby, A. W. Dean, H. K. Bishop, and A. H. Blanchard; this Committee to be added to later if the Board deems it necessary."

The Secretary reported the completion of the new tier of book stacks for the Library.

Applications were considered and other routine business transacted.

Adjourned.

* See page 232 for resolution adopted by the Society.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

June 2d, 1909.—8.30 P. M.—Two papers will be presented for discussion as follows: "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.; and "Caisson Disease and Its Prevention," by Henry Japp, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for April, 1909.

September 1st, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Concrete Piles," by Howard J. Cole, M. Am. Soc. C. E., and "Hydro-Electric Power in Canada," by Cecil B. Smith, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

J. W. Ellis,

A. W. Dean, J. F. Stevens,
H. W. Hayes, S. E. Tinkham,
George A. Kimball, H. D. Woods.

A circular will be issued about June 1st, 1909, giving information relative to transportation, and further details of the programme.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) has been issued. There will be three additional volumes issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is

dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume of 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From April 13th to May 10th, 1909)

DONATIONS*

SANITATION AND SANITARY ENGINEERING.

By Wm. Paul Gerhard. Second Revised and Enlarged Edition of "Sanitary Engineering." Cloth, 7½ x 5 in., 6 + 175 pp. New York, The Author, 1909. $1.50 net.

In this edition of "Sanitary Engineering" the title has been changed to "Sanitation and Sanitary Engineering," and the work enlarged by additional matter to more than twice its original size. After defining the term, sanitary engineering, the author describes briefly the course of study and special training necessary to the practice of this branch of the profession, including the courses given at the Massachusetts Institute of Technology and Cornell and Columbia Universities. This is followed by a general and condensed review of the various kinds of work and problems with which the sanitary engineer has to deal. The Contents are: Sanitary Engineering—The Profession and Its Practice; The Work of the Sanitary Engineer in Time of Epidemics, in Time of War, and in Sudden Calamities in Civic Life; A Half Century of Sanitation (1850-1900); Sanitation in Greater New York: Sanitation in Russia; Reference to Other Works of the Author Dealing More in Detail With Problems of Sanitary Engineering.

BEITRAG ZUR BERECHNUNG UND AUSFÜHRUNG DER STAUMAUERN.

Von Franz Kreuter. Cloth, 8½ x 5½ in., illus., 54 pp. München und Berlin, R. Oldenbourg, 1909. 2 marks, 40 pfennigs.

The author, in his prefatory remarks, states that this work constitutes an abstract of his lectures at the Münich Technische Hochschule, and is the result of many years' work. Other investigators have pointed out the defects in the old methods of calculating masonry dams, he says, but have put forward no new method, while he has fulfilled the new demands and found a solution of the problem which he offers to the profession. He claims, also, that while the present work is more complete and more accurate than his former treatise on the same subject, it is, at the same time, simpler and more comprehensive. The Contents are: Praktische Vorbedingungen; Theoretische Vorbedingungen; Entwicklung der Bestimmungsgleichungen; Anwendung; Schlussbemerkungen.

HANDBOOK OF TIMBER PRESERVATION.

By Samuel M. Rowe, M. Am. Soc. C. E. Souvenir Edition, Revised. Leather, 6 x 4 in.; illus., 402 pp. Chicago, Pettibone, Sawtelle & Co., 1909. $5. (Presented by the Author.)

This edition, which the author states in a "valedictory" to be the last, is enlarged by about seventy-five pages of new matter, the first part being practically unchanged. The additional matter includes tables on absorbent properties of timber, reprints of letters to the *Engineering News*, several pages on fungus culture, with plans for culture room, records of tie renewals, notes on the open-tank method of treatment, etc.

MINING METHODS IN EUROPE.

Written After a Visit to Many of the Metal and Coal Mines of Great Britain, France and Germany. By Lucius W. Mayer. Cloth, 9½ x 6 in., illus., 9 + 169 pp. New York and London, Hill Publishing Company, 1909. $2.50 net.

This volume describes what the author characterizes as the more or less distinct and often extraordinary systems of mining practised in various countries of Europe. The preface speaks of the scarcity of mining literature, especially on the subject of the method of underground attack, and states that this work is intended to help fill this lack, this feature, in every case, being explained in detail. In Part I, Mining in Great Britain, the chapter headings are: Mining in Great

*Unless otherwise specified, books in this list have been donated by the publisher.

Britain; Parkside Mine, Cumberland; Creator Moor Mine, Cumberland; Ullcoat Mine, Cumberland; Hodbarrow Mine, Cumberland; Longwall Methods of Mining Flat Seams; Seaton-Delaval Colliery, Northumberland; Chopwell Colliery, Northumberland; Kimblesworth Colliery, Northumberland; Watnall Colliery, Midland District; Coal Mining in South Staffordshire; Spawood Mine, Yorkshire; Park Pit, Yorkshire; Oakley Slate Quarries, North Wales. In Part II, Mining in Germany, France and Other Countries, the chapter headings are: Lead Mining in Mechernich, Prussia; The Flushing System; Coal Mining in Silesia; Flushing System at Gute Hoffnung Mine, Germany; Ilsede Hütte Mine, Peine, Germany; Coal Mines of Carmaux, France; Notes on Mining, Timbering, and Lighting.

TRIGONOMETRISCHE LÄNGENBESTIMMUNG GEODÄTISCHER GRUNDLINIEN.

Von A. Tichy. Paper, 11 x 7½ in., illus., 67 pp. Wien, Der Oesterreichische Ingenieure-und Architekte-Verein, 1909.

RAILROAD STRUCTURES AND ESTIMATES.

By J. W. Orrock. Cloth, 9 x 6 in., illus., 6 + 270 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $3 net.

The intention of this work, as set forth in the preface, is to cover in a brief and concise form the numerous subjects which enter into the engineer's estimates of railroad building, for the purpose of ready reference as to general construction and cost, on a business rather than on a technical basis. It is also stated that data to suit all conditions being impossible, the weights, quantities, and costs are given in detail in most instances. In many of the tables blank spaces are left for the addition by the user of the work of his own prices and records. The Contents are: Track Material; Fences, Gates, Sign Posts, Road Crossings and Guards; Culverts; Bridges; Buildings; Water Stations; Shops; Specifications and Contracts; Estimating Notes; Index.

Gifts have also been received from the following:

Allen, W. F. 1 pam.
Am. Electrochemical Soc. 1 vol.
Am. Iron and Steel Assoc. 1 pam.
Am. Min. Congress. 1 vol.
Am. Ry. Eng. and M. of W. Assoc. 2 pam.
Am. Soc. of Municipal Impvts. 1 bound vol.
Bandy, James M. 2 pam.
Barschall, Max. 1 pam.
Bengal, India-Irrig. Dept. 1 bound vol.
Black, W. M. 18 bound vol., 1 vol., 3 pam.
Bombay Presidency-Public Works Dept. 1 bound vol.
Brooklyn, N. Y.-Borough President. 2 bound vol.
California-State Harbor Commrs. 1 pam.
Canada-Board of Ry. Commrs. 1 vol.
Canada-Dept. of Mines, Mines Branch. 1 bound vol.
Canada-Geol. Survey. 4 pam.
Cleveland, Cincinnati, Chicago & St. Louis Ry. Co. 1 pam.
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Copper Range Consolidated Co. 1 pam.
Delaware & Hudson Co. 1 pam.
Delaware Coll. 1 pam.
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Ericson, John. 3 bound vol.
Grand Rapids & Indiana Ry. Co. 1 pam.
Greene Consolidated Copper Co. 2 pam.
Grover, O. L. 1 pam.
Harvard Univ. 1 vol.
Houston, A. C. 5 pam.
Ill.-State Geol. Survey. 1 bound vol.
Iowa Street and Interurban Ry. Assoc. 3 bound vol.
Lawrence, Mass.-Water Board. 1 pam.
Lea, Samuel H. 1 pam.
Lehigh Univ. 1 pam.
Little Falls, N. Y.-Board of Public Works. 1 pam.
Long Island R. R. Co. 1 pam.
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Mass.-Board of Gas and Elec. Light Commrs. 1 vol.
Mass.-Board of Met. Park Commrs. 1 bound vol.
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New York State-Museum. 1 bound vol.
New York State-Public Service Comm., 2d Dist. 1 pam.
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Cours de Ponts Métalliques Professé à l'École Nationale des Ponts et Chausées. Par Jean Résal. Tome 1. Encyclopédie des Travaux Publics. Ch. Beranger, Paris, 1908.

Handbuch der Ingenieurwissenschaften. Vol. 3, Part IV. Die Baumaschinen. Von F. Linke. Zweite vermehrte Auflage. Wilhelm Engelmann, Leipzig, 1908.

Disposal of Liquid Refuse from Distilleries. Sixth Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) May Properly be Adopted. Wyman & Sons, Ltd., London, 1909.

San Diego, a Comprehensive Plan for Its Improvement. By John Nolen. Geo. H. Ellis Co., Boston, 1908.

The Gas Engine. By Forrest R. Jones. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

Hydraulique Fluviale. La Forme du Lit des Rivières à Fond Mobile. Par L. Fargue. Encyclopédie des Travaux Publics. Gauthier-Villars, Paris, 1908.

An Elementary Manual of Radiotelegraphy and Radiotelephony for Students and Operators. By J. A. Fleming. Longmans, Green & Co., London, 1908.

SUMMARY OF ACCESSIONS

(From April 13th to May 10th, 1909)

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Total	173

MEMBERSHIP

ADDITIONS

(April 14th to May 11th, 1909)

MEMBERS		Date of Membership.
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BEATTY, PHILIP ASFORDBY. Civ. Engr., Water Dept., 3616 Fairview Ave., Forest Park, Baltimore, Md.	Assoc. M. M.	Nov. 6, 1907 May 4, 1909
BLANCHARD, ARTHUR HORACE. Assoc. Prof. of Civ. Eng., Brown Univ.; Deputy Engr., State Board of Public Roads of Rhode Island, East Side Station, Providence, R. I.	Jun. Assoc. M. M.	Jan. 2, 1900 April 5, 1905 May 4, 1909
CHOATE, JOSEPH KITTREDGE. Cooperstown, N. Y.		May 4, 1909
CLARKE, THOMAS CURTIS. Care, Lackawanna Steel Co., 2 Rector St., New York City		May 4, 1909
CRANDALL, CHARLES LEO. 304 Security Bldg., Galveston, Tex.		May 4, 1909
CRANE, CLARENCE AUSTIN. Civ. and Cons. Engr., 21 Park Row, New York City	Assoc. M. M.	Mar. 5, 1902 May 4, 1909
GRANBERY, JULIAN HASTINGS. 20 Broad St., New York City	Jun. Assoc. M. M.	Sept. 6, 1898 Sept. 2, 1903 April 6, 1909
HAMMOND, CHARLES ADRIAN. 301 South 4th Ave., Mt. Vernon, N. Y.		April 6, 1909
HOWELL, GEORGE PIERCE. Maj., Corps of Engrs., U. S. A., U. S. Engr. Office, Manila, Philippine Islands		Feb. 2, 1909
JARRETT, EDWIN SETON. The Foundation Co., 115 Broadway, New York City	Assoc. M. M.	May 1, 1895 April 6, 1909
MILES, JOHN WILEY. Care, American Club, City of Mexico, D. F., Mexico	Assoc. M. M.	Oct. 2, 1901 April 6, 1909
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PEABODY, WILLIAM WELCOME. Asst. Engr., Board of Water Supply, New York City, 24 Summit Ave., White Plains, N. Y.		April 6, 1909
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POND, HENRY OTIS. Tenafly, N. J.		May 4, 1909
RITTENHOUSE, WALTER BRITTON. 430 College Ave., Canon City, Colo.		April 6, 1909
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	Date of Membership.
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TAYLOR, HOWARD FLANDERS. Locating Engr., Central of Georgia Ry., 800 Brighton Ave., Kansas City, Mo. — Jun.	Feb. 28, 1899
— Assoc. M.	June 6, 1906
— M.	April 6, 1909
TENNEY, WILLIS ROBINSON. Room 12, Municipal Bldg., Brooklyn, N. Y.	May 4, 1909

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— Assoc. M.	April 6, 1909
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CAMPBELL, HENRY AVERY. 519 Monadnock Bldg., San Francisco, Cal. — Jun.	April 4, 1905
— Assoc. M.	April 6, 1909
ELLIOTT, JAMES WILLIAM. 41 So. Prospect St., Burlington, Vt.	April 6, 1909
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GREEN, JAMES COWAN. 68 N. Allen St., Albany, N. Y.	April 6, 1909
GREENE, WILLIAM STEWART. 284 Redmond St., New Brunswick, N. J.	May 4, 1909
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JONES, SAMUEL REYNOLDS. Care, J. G. White & Co., 43 Exchange Pl., New York City.	April 6, 1909
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MALCOLM, CHARLES WESLEY. Asst. Prof. of Structural Eng., Univ. of Illinois, 1001 W. Illinois St., Urbana, Ill. — Jun.	Sept. 4, 1906
— Assoc. M.	April 6, 1909
PICKWICK, EDWIN JOB. Parker Bldg., Schenectady, N. Y.	Jan. 5, 1909
PITKETHLY, DAVID THOMAS. 330 Etna St., Brooklyn, N. Y.	April 6, 1909
PLOGSTED, WALTER JOHN. Res. Engr., Gen. Ry. Signal Co., 527 Fifth Ave., New York City. — Jun.	Oct. 6, 1903
— Assoc. M.	May 4, 1909
REIMER, FREDERIC ADAMS. City Engr., 51 North Maple Ave., East Orange, N. J.	April 6, 1909
STEVENS, ELIHU WILLIAM. Poplar Ave., Hackensack, N. J.	April 6, 1909
STEVENS, HAROLD LYELL. Pres., The H. L. Stevens Co., 63 North Forsyth St., Atlanta, Ga.	April 6, 1909
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TAYLOR, WILLIAM PURVES. Engr. in Chg. of Philadelphia Municipal Testing Laboratories, 318 City Hall, Philadelphia, Pa..	Jun.	Oct. 7, 1902
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	Assoc. M.	April 6, 1909
TURNER, NATHANIEL PARKER. Care, Juragua Iron Co., Firmeza, Santiago, Cuba.....		Dec. 1, 1908
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JUNIORS

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APPEL, HARRIS ARKUSH. Box 504, Colorado Springs, Colo..	Mar. 2, 1909
BABCOCK, FRANKLIN. Care, Union Trust Co. of N. Y., 80 Broadway, New York City.....	Mar. 2, 1909
BILYEU, CHARLES SMITH. Care, P. T. & T. R. R. Co., 10 Bridge St. (Res., 401 West 117th St.), New York City.....	Dec. 1, 1908
COLLAR, WILLIAM FRANKLIN. Supt., The Foundation Co., Ishpeming, Mich.....	April 6, 1909
FELGENHAUER, FRANK JOHN. 1439 Dean St., Brooklyn, N. Y.	May 4, 1909
FOSS, JAMES CALVIN, JR. Asst. Engr., Hawaiian Irrig. Co., Kukuihaele, Hawaii.....	April 6, 1909
GILMORE, ALVIN LEROY. 318 West 57th St., New York City.	April 6, 1909
GOULD, CHESTER MASON. Cold Spring on Hudson, N. Y....	April 6, 1909
HAMILTON, EDWARD PARMELEE. Asst. Engr., Cuban Eng. & Contr. Co., Monte 15, Havana, Cuba.....	April 6, 1909
HAWLEY, CHARLES BURRIDGE. 2048 E. 115th St., Cleveland, Ohio.....	May 4, 1909
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WARNER, JAMES MADISON. 6339 Yale Ave., Chicago, Ill....	April 6, 1909
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* Owing to an error Colonel Bixby's address was entered in *Proceedings* for April, 1909, as 420 Bumiller Bldg., Los Angeles, Cal. The present address is correct.

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MOSS, ROBERT EDWARD. Structural Engr., 126 Liberty St., New York City.

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SWICKARD, ANDREW. 225 Adams St., San Jose, Cal.

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WAGNER, FRED J. First Res. Engr., Dept. of State Engr. and Surv., Syracuse, N. Y.

WHITE, FRANK GEORGE. Office of City Engr., Mechanics Bank Bldg., San Francisco, Cal.

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WILLIAMS, SAMUEL WALTER. 3337 Garfield St., Kansas City, Mo.

WOODWARD, EDWIN CARLTON. 1500 Cooper St., Fort Worth, Tex.

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BREUCHAUD, JULES ROWLEY. 290 Broadway, New York City.

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CONNER, RALPH MELVIN. Asst. Engr., U. S. Reclamation Service, Babb, Mont.

FORTNEY, CAMDEN PAGE. Care, Administration Bldg., Gatun, Canal Zone, Panama.

GAIGER, FRANK MILLARD. Katonah, N. Y.

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GREATHEAD, JOHN FRANCIS. Leveler, Barge Canal Office, Delta, Oneida Co., N. Y.

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GROSS, CHARLES AARON. Care, Bethlehem Steel Co., 111 Broadway, New York City.

HANAVAN, WILLIAM LAWRENCE. Asst. Engr., Board of Water Supply, New York City, Walden, N. Y.

HASELWOOD, FRED WILLIS. Res. Engr., Western Pacific Ry., 1st and Adeline Sts., Oakland, Cal.

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HITT, RODNEY. Assoc. Editor, *Electric Railway Journal*, 239 West 39th St., New York City.

HODGMAN, BURT BRADLEY. 61 Park Row, New York City.

HOLBROOK, ARTHUR RAYMOND. Asst. Engr. Designer, Board of Water Supply, P. O. Box 311, New Paltz, N. Y.

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KILKENNY, TOBIAS DILLON. 930 Pine St., San Francisco, Cal.

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DEATHS.

BARBER, AMZI LORENZO. Elected Fellow, March 19th, 1886; died April 18th, 1909.

MAYER, AUGUST. Elected Associate Member, June 1st, 1892; died March 25th, 1909.

MOULTON, MACE. Elected Member, June 4th, 1884; died April 27th, 1909.

RICE, LEWIS FREDERICK. Elected Member, January 2d, 1889; died April 13th, 1909.

ROSEWATER, ANDREW. Elected Member, October 3d, 1883; died April 17th, 1909.

UBSDELL, JOHN ARNOLD. Elected Associate, June 7th, 1905; died April 23d, 1909.

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(April 13th to May 10th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
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(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
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(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
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(51) *Deutsche Bauzeitung,* Berlin, Germany.
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(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
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(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
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Methods and Cost of Constructing Frame Cottages and Service Buildings for the Camaguey, Cuba, Waterworks.* Henry A. Young. (86) Apr. 14.
Some Methods in Competitive Reinforced Concrete Design. Ernest McCullough. (Abstract of paper read before the Armour Inst. of Tech.) (14) Apr. 17; (86) Apr. 14.
The Reconstruction in Reinforced Concrete of the Large Manufacturing Plant of J. B. King & Co., Staten Island, N. Y.* Arthur G. Hoadley. (13) Apr. 22.
Guest's Law of Combined Stress.* C. A. Smith. (11) Apr. 23.
The Building Exhibition. (12) Serial beginning Apr. 23.
Corrosion of Iron and Steel. Edward Crowe. (Abstract of paper read before the Cleveland Inst. of Engrs.) (12) Apr. 23.
Tables for Calculating Inertia of Built-up Girders.* H. F. Harris. (47) Apr. 23.
The Knickerbocker Trust Building Substructure.* (14) Apr. 24.
The Cummins Station Warehouse, Nashville, Tennessee.* (13) Apr. 24.
Some Experiments on Impact.* J. E. Sears. (11) Serial beginning Apr. 30.
The Dangers of Breeze Concrete.* (11) Apr. 30.
A Large Concrete and Timber Coal Pocket.* (14) May 8.
Foundations of the Northwestern Railway Terminal, Chicago.* (14) May 8.
Influence du Traitement Thermique sur les Déformations Linéaires des Aciers. Demozay. (93) Apr.
Ergänzungen zu der Berechnung der Betoneisen-Konstruktionen. Heinrich Pilgrim. (81) Vol. 3, 1909.
Bestimmung der Gesamtbasizität des Zementes. Eduard Toth. (80) Serial beginning Apr. 10.
Das Neue Theater in Kopenhagen.* E. Suenson. (78) Apr. 23.
Erdbebensichere Bauten.* F. von Emperger. (78) Apr. 23.

Water Supply.

The Status of the Wave Motor.* James T. Barkelew. (64) Apr. 13.
Irrigation Works in the Hawaiian Islands.* M. M. O'Shaughnessy, M. Am. Soc. C. E. (13) Apr. 15.
The Early Construction Methods at the Plant of the Central Georgia Power Company.* (14) Apr. 17.
Sedimentation of the Water-Supply and its Relation to Typhoid Fever in Washington, D. C.* Francis F. Longley. (13) Apr. 22.
The Valuation for City Purchase of the Property of the Waterloo (Ia.) Water-Works Co. A. Marston, M. Am. Soc. C. E. (13) Apr. 22.
The Garden City Pumping Plant.* F. W. Hanna. (14) Apr. 24.
Swedish Hydro-Electric Power-Plants.* John Geo. Leigh. (10) May.
Impurities Causing Scale and Corrosion. J. C. William Greth. (Abstract of paper read before the Amer. Inst. Chem. Engrs.) (94) May.
The Design of Steel Pipes for Hydro-Electric Plants.* Richard Muller. (14) May 1.
The High Head Hydro-Electric Development at Loch Leven, Scotland.* J. Murray S. Culbertson. (14) May 1.
Recherches et Observations Récentes sur le Belier Hydraulique.* Goupil. (43) Jan.
Les Nouveaux Filtres à Sable non Submergé système de Mm. le Docteur P. Miquel et Mouchet.* Félix Marboutin. (32) Feb.
Calcul des Tuyaux en Béton Armé soumis à des Efforts de Compression et de Flexion.* P. Caufourier. (33) Apr. 3.
Perturbations Produites par la Fermeture des Robinets-Vannes sur le Fonctionnement d'une Distribution d'Eau.* Henry Dide. (33) Apr. 24.
Die Wasserversorgungs-Anlage für Apulien. Dr. Brandau. (Paper read before the Arch.-u. Ing.-Verein zu Cassel.) (51) Serial beginning Apr. 10.

*Illustrated.

Waterway.

The Low Stages of Lakes Huron and Michigan.* C. E. Grunsky, M. Am. Soc. C. E. (54) Vol. 63.
The Floods of the Mississippi Delta: Their Causes and Suggestions as to their Control. William D. Pickett, M. Am. Soc. C. E. (54) Vol. 63.
The Improvement of the Ohio River.* William L. Sibert, M. Am. Soc. C. E. (54) Vol. 63.
The Colorado River Breach and Closure.* (12) Apr. 9.
The Proposed National Ship Canal in Scotland.* (11) Apr. 23.
Construction of a Sea Wall at Fort Morgan, Alabama.* (14) Apr. 24.
Extraordinary Ice Jams at Niagara Falls.* (13) Apr. 29.
The Present Condition of the Suez Canal.* (12) Apr. 30.
A Dredging Equipment in the Himalaya Mountains.* (95) May.
An Eight-Yard Dipper Dredge for the Cuban Government.* (95) May.
Design of Hulls for Hydraulic Cutter Dredges. E. N. Percy. (95) May.
The Bates Electrically Driven Hydraulic Dredger.* (95) May.
Lock No. 3, Cumberland River.* William W. Harts, M. Am. Soc. C. E. (13) May 6.
The Design of Cable Stations for River Measurements.* J. C. Stevens. (13) May 6.
Built-Up Concrete Slab Piles and Pile Protection.* (14) May 8.
New Municipal Piers in Baltimore Harbor.* (14) May 8.
Les Echelles à Poissons et leur Application aux Barrages de Meuse et d'Ourthe.* G. Denil. (30) Apr.
Errichtung eines Kais mittels Eisenbeton-Schwimmblöcken in Port Said.* Walther Stross. (78) Apr. 2.
Neuere Wehrbauten aus Eisenbeton.* F. W. Otto Schulze. (78) Serial beginning Apr. 2.
Vom Panamakanal.* Max Frhrn. von Wendland. (51) Serial beginning Apr. 7.

*Illustrated.

Vol. XXXV. MAY, 1909. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

April 21st, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 108 members and 53 guests.

A paper by C. S. Bissell, M. Am. Soc. C. E., entitled "The Maximum Weights of Slow Freight Trains," was presented by the Secretary, who also read a communication on the subject from Mr. L. A. Riley, 2d.

A paper by W. W. Crosby, M. Am. Soc. C. E., entitled "Sampittic Surfacing," was presented by the author and discussed orally by Messrs. A. H. Blanchard, Francis T. Smith, Philip P. Sharples, H. W. Clark, J. W. Howard, and the author. Written communications on the subject from Messrs. L. W. Page and Prevost Hubbard were presented by the Secretary.

W. W. Crosby, M. Am. Soc. C. E., offered the following resolution:

"*Resolved:* That the Board of Direction of the American Society

of Civil Engineers be requested to appoint a Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use."

The Resolution, being duly seconded, was adopted by a vote of more than 25 Corporate Members.

The Secretary announced the following deaths:

Lewis Frederick Rice, elected Member January 2d, 1889; died April 13th, 1909.

Andrew Rosewater, elected Member October 3d, 1883; died April 17th, 1909.

August Mayer, elected Associate Member June 1st, 1892; died March 25th, 1909.

Adjourned.

May 5th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, 132 members and 31 guests.

The minutes of the meetings of March 17th and April 7th, 1909, were approved as printed in the *Proceedings* for April, 1909.

The Secretary announced that in accordance with the Resolution adopted by the Society at its meeting on April 21st, 1909, the Board of Direction, at its regular meeting, May 4th, 1909, had appointed the following Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use: Messrs. W. W. Crosby, A. W. Dean, H. K. Bishop, and A. H. Blanchard.

A paper by Myron S. Falk, M. Am. Soc. C. E., entitled "A System of Cost Keeping," was presented by the author and discussed orally by Messrs. William M. Black, Robert B. Stanton, T. Kennard Thomson, Willard D. Lockwood, William B. Reed, and Guy W. Culgin.

Written communications on the subject from Messrs. P. L. Reed, W. C. Hammatt, and Emile Low, were read by the Secretary.

A paper by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., entitled "The Design of Elevated Tanks and Stand-Pipes," was presented by the Secretary, who also read communications on the subject from Messrs. Charles B. Burdick, and O. E. Selby. The paper was discussed orally by Messrs. William M. Black, and James M. Porter.

The Secretary announced the election of the following candidates by the Board of Direction on April 6th, 1909:

As Members.

John Bennington Berry, Chicago, Ill.
Oscar Sidney Bowen, Spokane, Wash.
William Howell Caldwell, Tampa, Fla.
Joseph Kittredge Choate, Cooperstown, N. Y.

THOMAS CURTIS CLARKE, New York City.
CHARLES LEO CRANDALL, Galveston, Tex.
HARRY HAWGOOD, Los Angeles, Cal.
THOMAS NOTTINGHAM JACOB, Herrin, Ill.
WILLIAM PRENTISS MORSE, Newton, Mass.
WILLIAM RODNEY PATTERSON, Chicago, Ill.
HENRY OTIS POND, Tenafly, N. J.
WALTER JUSTIN SHERMAN, Toledo, Ohio.
WILLIS ROBINSON TENNEY, Brooklyn, N. Y.
EDWIN DERICKSON VINCENT, Yuma, Ariz.

As Associate Members.

JOHN HOWARD ALEXANDER, St. Louis, Mo.
WALTER CHARNLEY, Rio Grande do Sul, Brazil.
FRANK LEMUEL CLAPP, Cornwall-on-Hudson, N. Y.
JOHN SEBASTIAN CONWAY, Portland, Ore.
BERTRAM EDWIN CORLETT, Seattle, Wash.
CHESTER CAWTHORNE DERBY, Richmond Hill, N. Y.
ARCHIBALD MARVINE GILBERT, Twin Falls, Idaho.
WILLIAM STEWART GREENE, New Brunswick, N. J.
RUSSELL THEODORE HARTMAN, Des Moines, Iowa.
WILLIAM SYLVESTER HAZELTON, Detroit, Mich.
LARS RASMUS JORGENSEN, San Francisco, Cal.
ALFRED BROUGHTON KISSACK, Leavenworth, Kans.
WILLIAM ARTHUR LAFLER, Rome, N. Y.
CLARENCE BOOTH LAMONT, Seattle, Wash.
WALTER FRANKLIN LINEBERGER, Torreón, Coahuila, Mexico.
THOMAS CLAWSON MARSHALL, Middletown, N. Y.
BERTRAND CLIFFORD MARTIN, Watertown, N. Y.
FRANK DANA NASH, Manila, Philippine Islands.
HARRY BRADFORD POND, New York City.
HARRY IVES SHOEMAKER, Puunene, Hawaii.

As Associates.

JOHN COFFEE HAYS, Visalia, Cal.
FRANK MAYHEW TALBOT, Glen Ridge, N. J.

As Juniors.

CHESTER DANIEL ASHBROOK, Seattle, Wash.
HARRY MONTIFIORE BERGMAN, New York City.
LYMAN EDGAR BISHOP, Denver, Colo.
MYRON CARLOS BURR, Vancouver, B. C., Canada.
STANLEY GARDNER CUTLER, Manila, Philippine Islands.
FRANK JOHN FELGENHAUER, Brooklyn, N. Y.

Robert Maximilian Feustel, Madison, Wis.
Alvero Charles Gregson, New York City.
Charles Burridge Hawley, Cleveland, Ohio.
Cyrus Pierce Howes, St. Louis, Mo.
Andrew Peach Rollins, Eagle Lake, Tex.
Harvey Hillyer Sims, Savannah, Ga.
Thomas Harrison Winchester, Macon, Ga.

The Secretary announced the transfer of the following candidates by the Board of Direction on May 4th, 1909:

From Associate Member to Member.

Philip Asfordby Beatty, Baltimore, Md.
Arthur Horace Blanchard, Providence, R. I.
Clarence Austin Crane, New York City.
Frank Tenney Darrow, Lincoln, Nebr.
Leonard Henry Davis, Sault Ste. Marie, Ont., Canada.
Frank Lawton Getman, Havana, Cuba.
Harry Thomas Paterson, Beaufort, N. C.
Samuel Payson Waldron, East Orange, N. J.

From Junior to Associate Member.

William Burton, Detroit, Mich.
William Hale Phillips, Spokane, Wash.
Walter John Plogsted, New York City.
William Purves Taylor, Philadelphia, Pa.

The Secretary announced the following deaths:

Amzi Lorenzo Barber, elected Fellow March 19th, 1886; died April 18th, 1909.

John Arnold Ubsdell, elected Associate June 7th, 1905; died April 23d, 1909.

Mace Moulton, elected Member June 4th, 1884; died April 27th, 1909.

Adjourned.

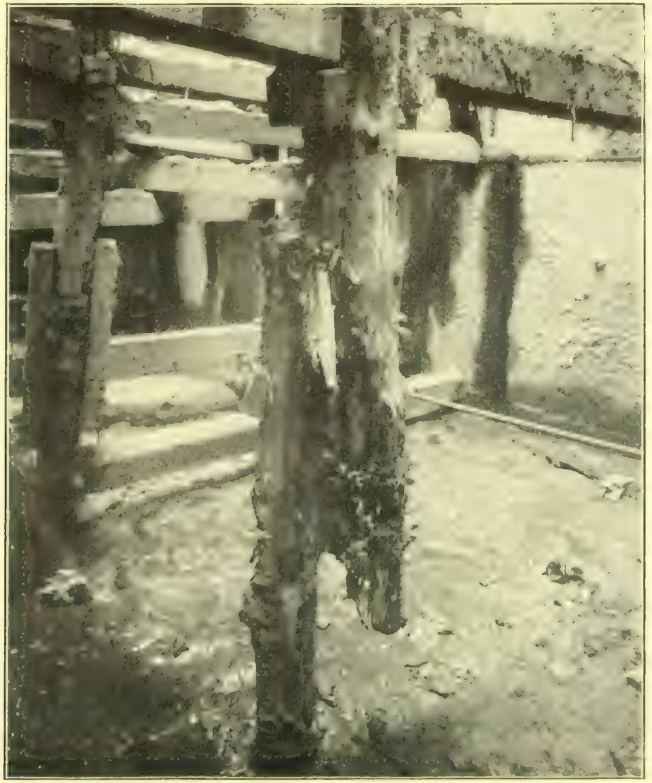

FIG. 1.—TIMBER PILES DAMAGED IN DRIVING: EXPOSED BY EXCAVATION.

FIG. 2. SUPERSTRUCTURE FOR OIL TANK COLUMNS SUPPORTED ON TYPE A OF THE MADE-IN PLACE CLASS.

amount, therefore the carrying capacity of the concrete pile is double that of the timber pile.

The method adopted in the United States to demonstrate the carrying capacity of a concrete pile has been to make an actual loading test of some of the piles by placing a platform on a pile, loading it with pig iron to double the guaranteed load, and, by a series of accurate levels, keeping watch for any settlement that may occur.

Some attempts have been made to calculate the supporting power of concrete piles by the formulas generally used with timber piles, but it is quite evident that none of these formulas will apply in cases where the concrete pile is jetted into place. When it is driven into position, however, a formula like "Ritter's," which takes into account the weight of the pile, can be used with good results. The "Ritter" formula is:

$$W = \left(\frac{h}{e}\right) \frac{Q^2}{Q+q} + Q + q,$$

in which W = The resistance to further penetration,
Q = The weight of the hammer, in pounds,
q = The weight of the pile, in pounds,
h = The height of the fall, in inches,
e = The penetration, in inches, by the last blow.

The concrete pile costs more per linear foot than the timber pile, but, in many installations, some items of construction, necessary with the timber pile, are eliminated entirely or reduced materially by the use of the concrete pile, so that the final result shows a considerable saving in time and money with the latter. For instance, where timber piles are used and the ground-water is several feet below the surface, it is customary to excavate to the water level and support the sides of the excavation with sheathing before driving the piles; then, after the piles are in place, they are cut off at the proper elevation, the pier or wall is built to the required height, and the excavation is back-filled. This proceeding is simplified greatly with the concrete pile foundation, where the pile is driven from the surface of the ground, without any preliminary excavation, and only requires a capping extending below the frost line to prepare it for the superstructure. Under these conditions, the concrete pile foundation saves on the items of excavation and depth of pier while it eliminates entirely the

items of sheathing, pumping, and back-filling, and at the same time accomplishes the result in a much shorter period of time.

The ideal location for a concrete pile is in filled areas where the water level is at considerable distance below the surface and the ground is of such a nature that it is necessary to carry the footings to a firm underlying stratum; here, the concrete pile can compete with the usual concrete or masonry pier, as, under these conditions, it will save most of the items mentioned in the example just given. In the construction of foundations for a large warehouse at Pittsburg, Pa., on the site of an old fill, from 20 to 30 ft. deep, where it was necessary to carry the foundations to a firm stratum, bids were received on two methods of construction: one being to carry the loads to the proper depth by concrete piers in open excavated pits, and the other to use concrete piles. The lowest time limit set by the open-pier method was 18 months; the work was actually done by the concrete-pile method in 76 days, thereby effecting a saving of more than 15 months in time alone, as well as a considerable amount of money.

Comparisons of the relative cost of concrete and timber pile foundations in fair-sized installations are not common, but an instance is cited in detail of a foundation at Annapolis, Md., which shows a saving in money of more than 50% on the cost of a timber pile foundation:

2 193 timber piles at $9.50	$20 833.50	
4 542 cu. yd. excavation at $0.40	1 816.80	
3 250 cu. yd. concrete at $8.00	26 000.00	
5 222 lb. I-beams at $0.04	208.88	
Shoring and pumping	4 000.00	
		$52 859.18
855 concrete piles at $20.00	$17 100.00	
1 038 cu. yd. excavation at $0.40	415.20	
986 cu. yd. concrete at $8.00	7 888.00	
		25 403.20
A saving of		$27 455.98

The saving in material was:

1 338 piles,

3 504 cu. yd. excavation,

2 264 cu. yd. concrete,
5 222 lb. I-beams, and
all shoring and pumping.

Although the concrete pile is cheaper under favorable conditions, the timber pile will be used for many years, especially in places where the ground-water level is near the surface and the building to be supported is so light and cheap as not to warrant an expensive foundation.

There are many kinds of concrete piles on the market. Most of these are patented and are the varieties in general use; they can be divided into two general classes, according to their method of construction: First, those made in place, where the pile hole is made by a mandrel and the concrete is dropped directly into the position it is to occupy as a concrete pile and sets in place; and second, those moulded or cast and allowed to harden thoroughly before being placed in position. For piles of this class the pile hole is not prepared in advance, but the pile is driven or jetted into place in a manner similar to that used with timber piles.

These piles all differ from each other in shape, being either straight or tapering, and in cross-section being square, polygonal or circular. These variations in form have led to considerable discussion as to the merits of different combinations, but it is the intention in this paper to mention in an impartial manner only the relative bearing powers of straight and tapering piles, without going into the comparative merits of the others, leaving that element to be brought out in the discussion, with the details of construction, cost figures, etc.

In the first class to be described there are two representative patented types. Both of these were first patented and used in the United States, and are distinctly American; consequently, installations of this class have been more numerous than of the second or moulded class. One of these, which for purposes of identification will be called Pile "A," is formed by driving to refusal a hollow steel pipe or form, of the proper length, $\frac{3}{8}$ in. in thickness of shell, 16 in. outside diameter, and of the same diameter throughout its entire length. This hollow form is driven by an extra heavily built pile-driver, which, in addition to the usual equipment, has a strong pulling device attached to the leads for the purpose of withdrawing the form from the ground after driving. The lower end of the form has a special jaw or shoe which

is closed during driving, in order to prevent the entrance of the surrounding materials into the form; in certain soils the "Alligator Jaw" point is used. This jaw is attached securely to the form by cable hinges, and is used over and over again. It is composed of two symmetrical wedge-shaped parts which are kept closed during driving by the pressure of the soil. Generally, a detachable cast-iron point is used, but this is more expensive than the "Jaw," as it is left permanently in place, and one must be supplied for each pile. The upper end of the form is reinforced with a strong band of steel shrunk in place and secured firmly to the form with countersunk rivets; this steel band also serves as a shoulder for the clamps which are attached to the pulling device. A steel driving head, sliding in the leads, with a circular projection on its lower face to fit into the top of the form, is lowered on the form, and not only serves to hold the top of the form firmly in place during driving, but also receives the impact of the hammer, from which it is protected by a rope mat.

When the form has been driven to refusal, the driving head is hooked to the hammer by a wire attachment, and both are hauled to the top of the leads and chocked there during concreting operations. A charge of concrete—about 5 cu. ft.—is dropped into the form, after which the entire form is raised about a foot by the pulling device; then a heavy iron rammer is dropped on the mass and by its impact opens the jaw and forces the concrete out into the space made by the end of the form. This procedure is repeated until the form has been entirely withdrawn from the ground and the space it occupied is completely filled with concrete, which hardens in place and becomes a concrete pile.

This method of filling is varied to meet varying conditions; occasionally, after the jaw has been opened, a sufficient quantity of concrete to make a complete pile is placed in the form and the form is steadily withdrawn; this plan, under certain sub-surface conditions, gives a better shaped pile, because, under the first method, the jaws have a tendency to close partly on account of the excess of pressure at each stop of the form, thus making a pile of irregular section; this is avoided by giving a regular and steady movement to the form. The concrete is made quite wet, of a 1:2:4 mixture, using $\frac{3}{4}$-in. stone, and, when placed in the form in mass, is heavier than the surrounding soil. Thus it is more than able to balance the pressure, and the soil has

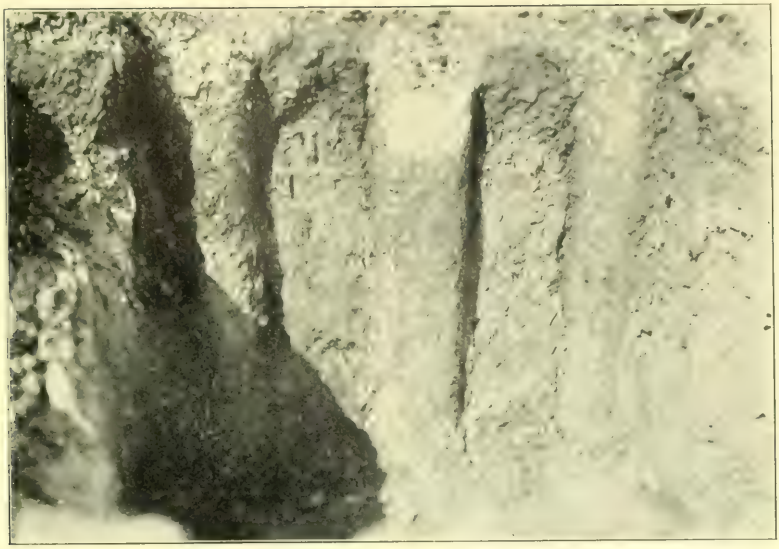

FIG. 1.—TYPE A. MADE-IN-PLACE CLASS: PILES EXPOSED FOR INSPECTION

FIG. 2.—MOULDED PILES, SIXTH STREET VIADUCT, KANSAS CITY.

no tendency to cut off any section of the pile, as the excess pressure serves to force the concrete into the soil.

In very hard driving, heavy forms, even with a $\frac{3}{4}$-in. shell, have been buckled at the top, and one can judge from this what the condition of a timber pile would have been under similar conditions. In one case the form was driven into the lower stratum so firmly that the pulling device could not withdraw it, so it was filled with concrete and left permanently in place.

It is claimed for piles of this type that the fresh concrete, which comes into direct contact with the surrounding material, enters into all its irregularities and unites with it, thus forming a rough exterior surface, which gives the pile a large amount of frictional resistance; and, further, as practically all the resistance to driving is confined to the point, the greater part of the compression takes place where the pile needs it most in order to transfer its load as a column.

In locations where the strata penetrated by the pile are in a fluid or semi-fluid condition, and have not enough consistency to serve as a mould to retain the concrete in shape, piles of this type could not be used without material modification. In one case of this nature, after the driving form had passed through the soft soil and had been driven to refusal into the firm underlying stratum, the initial charge of concrete was inserted and the point was forced open; then an auxiliary form of thin No. 20 metal, of slightly smaller diameter than the driving form, was dropped into place, and the required reinforcing was adjusted. Then the auxiliary form was completely filled with concrete, and the driving form was withdrawn, leaving the thin shell in position to act as a mould for the concrete. The foundations for a public utility building on Staten Island, N. Y., were constructed in this manner, the resulting pile being one which had its point embedded in the bearing stratum 8 ft. and was 16 in. in diameter in this portion, while the column part of the pile was 12 in. in diameter, and was enclosed in a shell of No. 20 metal. These piles stood successfully the test loading.

The length of piles of the ordinary type driven by this system is between 20 and 25 ft., but piles 48 ft. long have been used. The largest installation of this type is in the foundation of the Pittsburg Terminal Warehouse, Pittsburg, Pa., where about 6 000 piles, from 30 to 45 ft. long, were placed.

The other leading type of this made-in-place class, Pile "B", differs considerably from the one just described. It is tapering in shape, and each pile has a steel shell, to prevent the earthy material from mixing with the concrete, this shell being left permanently in position. A heavy pile-driver, usually on a turn-table frame for facility of operation, is used to drive a tapering collapsible form or core into the ground to make the pile hole; this core weighs about 3 tons, and is built in three longitudinal sections with provision for horizontal movement that permits the sections to be expanded or collapsed. Before beginning operations the core is suspended in the leads, with the point about 8 ft. above the ground to allow clearance for the sections of casing to be slipped on. The casing is made from sheets of No. 20 metal, 8 ft. long, which are formed into the required shape, the edges being bent to connect together and retain the shape of a truncated cone. These are of various sizes, to fit the tapering core, and are slipped on in regular order with a considerable overlap to each section, enough sections being used to cover the core completely to the length required. Occasionally, the joint between the sections is covered with an extra section to make a tighter joint. When this is done, a pressed-steel boot or thimble is forced on the point of the core overlapping the last section of casing. The core is now ready for driving. It is slung back into the leads, lowered carefully to the proper location on the ground, and is then driven to refusal by a steam hammer weighing about 3 tons. When the core has been driven to the desired depth, it is collapsed and withdrawn, leaving the steel casing in the ground as a lining for the pile hole; this is immediately filled with 1:2:4 concrete, using $\frac{3}{4}$-in. stone or gravel, mixed rather wet, which sets and forms the concrete pile. The concrete cannot be placed until the core has been withdrawn. In practice, it has been found that these protecting shells, being sectional, are not tight, but permit the entrance of water and quicksand which has to be siphoned out before the concrete is placed. When installed in locations where the shells are subjected to great hydrostatic pressure, the casing is sometimes collapsed and must be re-driven. In very hard driving in boulder formation, the core has been injured so severely as to be put out of commission and has had to be replaced by another.

It is claimed for this pile that its strong tapering shape, 20 in. at the butt and 8 in. at the point, distributes the load throughout the

entire length of the pile and compresses the soil to such an extent that the full bearing of the upper layers is fully utilized, and also, that the metal casing prevents the surrounding soil from mixing with and weakening the concrete. The Hospital Buildings of the United States Immigration Commission, at Ellis Island, New York Harbor, were installed by this method.

These two varieties of the made-in-place class have been more popular than any others in the United States in building foundations. They are reinforced occasionally with rods which are put in position as the concrete is deposited, or single rods may be churned into place through the wet concrete. The rods are left with their ends projecting above the concrete 1 ft. or more, in order to assist in bonding the capping to the piles. This made-in-place class is not adapted for use in docks, piers, wharfs, etc., without very material modifications of the typical pile described.

The installation of these piles has not always been a success; there have been failures, due to various causes, from which much has been learned. In one case all the subsoil conditions were favorable, the driving was satisfactory, and everything indicated a very successful piece of work; but when the physical test was applied the piles failed to support the load. On investigation it was found that the fault lay with the cement. One of the standard brands had been used, but it had been accepted without test, and proved worthless. In other cases difficult sub-surface conditions were met, each of which required some change in operations; but, at the present time, an unsuccessful installation is rare when all the conditions of the problem are carefully studied and investigated.

No single type of concrete pile will fit all locations without some change to suit the site, and, before determining what particular pile shall be used, a very careful survey of the subterranean strata should be made by taking test borings in several parts of the proposed site. Good wash borings will generally furnish this information, but core borings are more reliable though somewhat more expensive. A few years ago it was customary to drive test piles, in order to determine what length of pile to specify in the contract, but this plan was usually rather expensive, unless a pile-driver was on the site. In one instance four timber test piles cost $200 each, on account of the expense of moving the pile-driver to the site. Test borings should show the

various strata, their thickness, etc., and the elevation of the ground-water; the strata will be of different thicknesses and elevations in different parts of the site, and the borings should provide the information necessary to determine the length of pile, the type of pile to be used, and on which stratum to place the load. Neglect of these precautions is responsible for some of the settlements which have occurred in pile foundations, both timber and concrete. One instance may be cited where a 10-ft. fill had been placed on a swampy piece of ground, the bottom of which was 40 ft. below. To have reached this hard bottom, the piles should have been 40 ft. long, but a tapering pile of about two-thirds that length was used, and, for its support, relied on the filled layer and the spread of the concrete capping, and thus made what might be termed a floating foundation. This foundation settled slightly at first, and continued to go down somewhat each year until the total settlement is now more than 3 in.

One advantage of made-in-place piles over moulded piles is that, as they harden in the place where they are to be used, a small foundation could be completed before the moulded piles would be ready to drive, thus effecting a small saving in time.

In the construction of the foundations of a garage at Albany, N. Y., an unpatented type of this class was used, which serves to show one of its possible variations. The original surface of the site was a stiff clay and had been covered to the depth of 15 ft. by clay from adjoining excavations; thus it was necessary to carry some of the load to the original surface. This was accomplished by excavating holes, 10 in. in diameter, through the overlying fill to the original surface, using a screw post-hole digger, and then filling the holes with fresh concrete, the clay filling being stiff enough to retain its position during the operation. The piles were spaced at 3-ft. centers, two staggered rows being placed under the walls, while the pier loads were supported by clusters similarly spaced. The total length of piles was 4 779 ft.; the labor of digging the holes cost 20 cents per lin. ft., and the concrete $7 per cu. yd. in place, or about 13 cents per lin. ft., making a total of 33 cents per lin. ft. of completed pile. In pile work there is usually quite a large charge for plant installation and dismantling, but in this instance this was reduced to a minimum, which accounts for the low unit cost. No compression of the pile hole was made, but the supporting power of the pile was assisted materially by the capping, which acted as a spread footing.

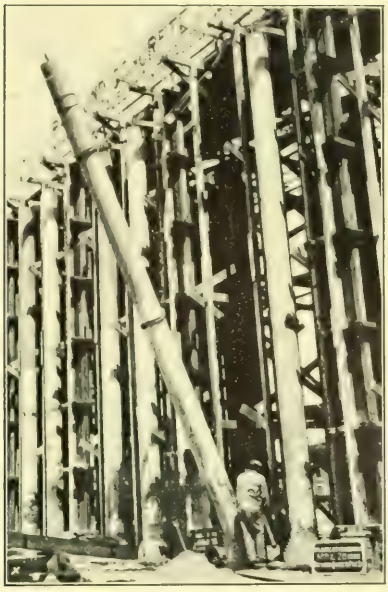

FIG. 1.—METHOD OF MOULDING AND LOWERING COMPLETED PILES, ATLANTIC CITY BOARDWALK.

FIG. 2.—VERTICALLY MOULDED PILE, ATLANTIC CITY BOARDWALK.

Another type of this class has been used in the clay soils around Boston, Mass., where the clay was overlaid by some 20 ft. of sand and silt. Through this an 8-in. casing was forced into the clay and the materials were washed out. Then a chambering device was used to expand the footing of the pile by cutting away the clay, which was also removed by washing. In this way the area of the footing of the pile was made 16 times as great as that of the pile itself—a very effective method of obtaining a spread footing at considerable depth at a moderate cost. This type of pile has been patented.

Just previous to the construction of the pneumatic foundations of the Trinity-United States Realty Buildings, at Broadway, Cedar Street, and Trinity Place, New York City, the foundations of three columns of the Sixth Avenue Elevated Railroad, located on the curb line of the Trinity Place front, were carried to hardpan, 75 ft. below the curb, by using steel-encased concrete piles. This was done in order to avoid any possible settlement of the railroad structure during the sinking of the caissons, which were to be carried to bed-rock. The original piers of these columns were founded on the sand at about ground-water level, 12 ft. below the curb. Below this footing elevation there was a thick stratum of quicksand extending to the hardpan, the latter resting directly on the bed-rock. The transverse girder supported by the first column was shored up, and the column and its pier were removed and then restored completely before any work was done on the next column. Over the close-sheeted excavation made for the removal of the old pier there was used a short pile-driver which could be operated under the overhanging girder, and this was assisted by a water jet, to force a 12-in. standard, black, wrought-iron pipe, in random-length sections, to the hardpan. When the first section had been worked down to the bottom of the sheeted excavation, another section was joined to it with outside couplings, the threads on the ends of the pipes being cut long enough to allow the ends of the sections to butt properly. The lower end of the bottom section was protected in most cases by an open driving shoe, which made it easier to sink the pile shell than with a cone-shaped, closed point, as it allowed the free use of the jet to soften up the material. The upper end of each section was protected from the direct impact of the hammer by a driving head which screwed into its coupling and butted on the pipe. This driving head had a slotted opening in its side through which the jet was worked during the operation of the hammer.

Most of the material from the inside of the pipe was removed by the jetting process; any that remained was taken out by an ordinary sand pump. There was no difficulty in getting out all the material after the pipe had reached the hardpan, the depth being checked by steel tape measurements, and the interior of the pipe inspected by the aid of a lantern. The pipe was then filled with 1:2:4 concrete. Five piles were thus placed for each pier, one near each corner, and the fifth in their midst directly beneath the center of the column. The piles extended 5 ft. into the mass of concrete which formed the new pier, and was 8 ft. square at the base and stepped in to meet the column base at the curb level.

A similar pile has been used in the underpinning of buildings in New York City. It is placed directly beneath the wall by cutting in the latter a vertical opening, from the starting elevation and through the foundation courses, of sufficient width and depth to leave ample working space around the pipe. The sections are forced into the soil by hydraulic jacks assisted with water jets, using the remainder of the wall for reaction purposes.

These piles are of various diameters, from 8 in. to 3 ft., in proportion to the loads to be carried, and the spaces are cleaned out in a manner similar to that just described, except that the largest sizes, in which there is space enough for a man to work, are cleaned by hand. The spaces are then filled with concrete under air pressure. When the pile is completed the load of the building is transferred to it by masonry and wedging stones. It is the most effective system of underpinning in use to-day, as it gives a positive assurance of the safety of the building, and at the same time does not occupy any space outside the wall, thus the contractor has a lot free from all obstructions.

Some of the tall buildings in New York City are supported on foundations of a patented type composed of steel-encased concrete piles, heavily reinforced throughout, and driven to place in a manner somewhat similar to that used in sinking other encased piles, except that a pneumatic hammer is used in connection with the water jet. The shell of this pile is generally $\frac{3}{8}$ in. in thickness and 12 in. in exterior diameter, the sections being joined by inside couplings. The reinforcing rods are inserted in sections, which are fastened together by an effective mechanical device, and form a continuous rod. The

FIG. 1.—TYPE B, MADE-IN-PLACE CLASS; METHOD OF MAKING CASING.

FIG. 2.—TYPE B, MADE-IN-PLACE CLASS; PILE EXTENDED ABOVE GROUND TO RECEIVE CAPPING.

earthy material in the shell is withdrawn, and is replaced by concrete, in the manner heretofore described. On account of the large area of steel in its cross-section, the Building Department of New York City has given this pile a much higher bearing value than any other, namely: 350 lb. per sq. in. on the concrete in compression and 4 200 lb. per sq. in. on the steel. The life of the steel in the exterior shell is somewhat doubtful, as it is subjected not only to the oxidizing influence of the ground-water, but also to the corrosive action of gas in the soil, and to stray electric currents; therefore it should not be considered as a permanent element in the strength of the pile.

The cost per linear foot of the made-in-place class varies with the type of pile, amount of reinforcing, the size of the contract, the accessibility of the work, and the sub-surface conditions. As no two pieces of work are alike, it is difficult to give more than a general idea of the cost. In the representative types of this class the cost will vary from 75 cents to $1.50 per linear foot of pile, depending on the foregoing conditions; the steel-encased varieties are more expensive, as the casing alone costs more than the above-mentioned price.

Concrete piles of this class should always be reinforced in order to provide for all cross-strains due to sub-surface conditions or those which may take place during construction, and while the pile is still green, particularly when the length of the pile is more than 15 ft. Piles of this class are better adapted to lengths of less than 30 ft., while, for greater lengths, a moulded type is more desirable, as there is doubt as to the homogeneousness of the concrete when it is dropped 30 ft. and cannot be thoroughly spaded in place.

In the second class of concrete piles—the moulded type—the patented varieties, like those of the first class, are more generally known than the unpatented. They also differ from each other in shape, cross-section, and method of reinforcement. The moulded pile was first developed in Europe, and is used extensively there, and also in the United States, for piers, docks, and similar structures, for which it is specially adapted. It is also being used in the foundations of buildings and other structures where subsoil conditions are favorable, and especially where long piles are desired. Piles of this class, more than 60 ft. in length, have been successfully driven, pulled out, and re-driven several times without showing any signs of weakness. All moulded piles are reinforced to stand the rough handling to which

they are subjected in transportation from the moulding yard to the site and in being placed in the leads, as they are sometimes hauled up by a line attached to the butt of the pile, instead of by a bridle which would lessen the cross-strain. The driving strains are very great, and it is thought by many that a pile with reinforcement should not be subjected to the impact of a hammer; but many piles have been driven in this way, and later pulled up and re-driven, without showing any damage by the hammer. In one instance an attempt was made to destroy a pile by hard driving, but the only effect was to mash or broom the butt of the pile for about 2 ft., the remainder being uninjured.

Most of the moulded piles are driven into place; but, when conditions are favorable, they are jetted, and in some varieties special provision is made for jetting by inserting a 2-in. pipe in the center of the pile. In city work it is seldom practicable to use the jet, on account of danger to the foundations of adjoining property. When

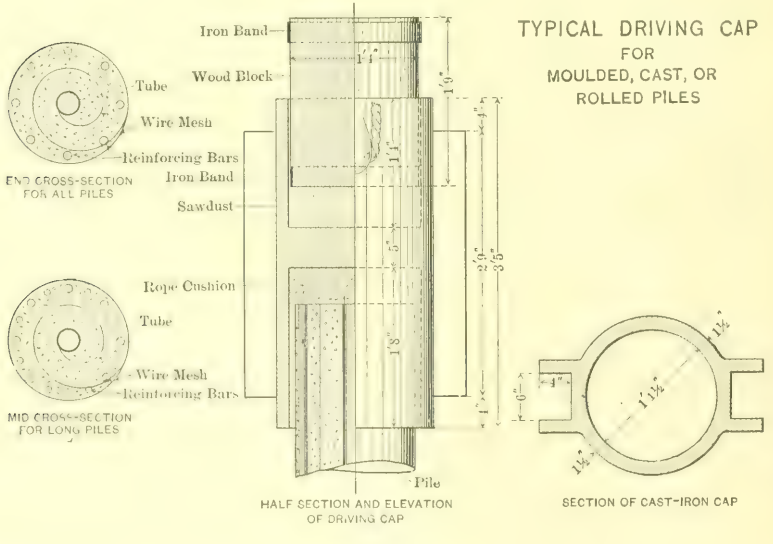

FIG. 1.

driven into place, some provision is made for cushioning the blow of the hammer. This is usually done by the insertion of a false pile of wood, acting on a sawdust or sand cushion, between the hammer

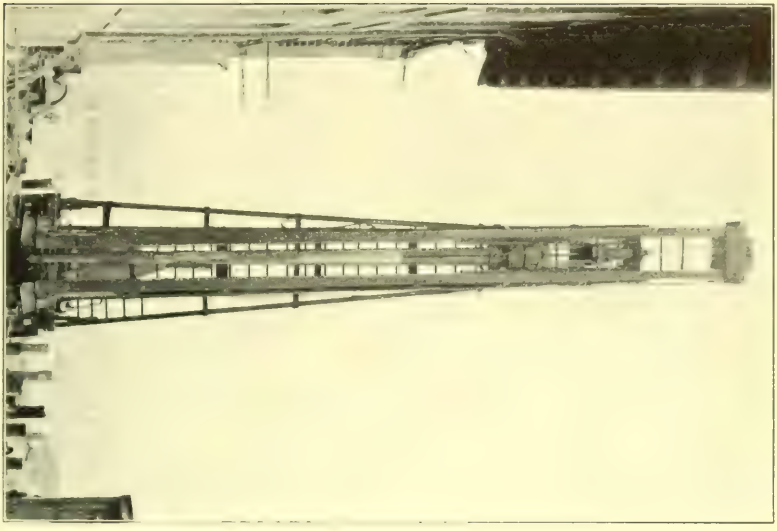

FIG. 1.—TYPE B. MADE IN PLACE CLASS. FORM ENCASED IN METAL, READY TO DRIVE.

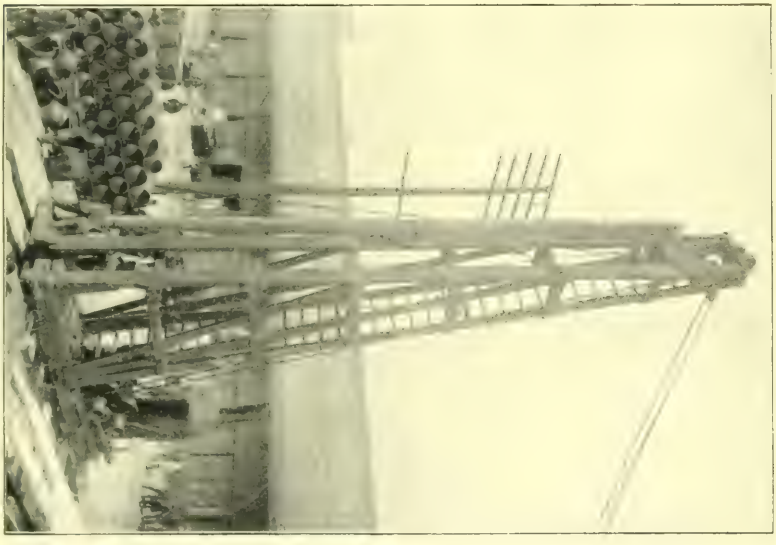

FIG. 2.—TYPE B. MOULDED CLASS. HAULING 61 FT. PILE INTO LEADS.

and the head of the pile, the whole driving head being held in position on the pile by an encircling metal cylinder held securely in place.

Piles of this class are moulded either horizontally or vertically, one variety being rolled instead of moulded or cast. The vertical system of moulding is the more expensive in time and labor, but, when thoroughly done, it produces a good pile. It is possible to effect considerable saving in forms alone by the horizontal system, as the upper portion or lid of the form for square or polygonal cross-sections can be omitted entirely, while the side pieces can be removed after 24 hours and used again, thus leaving only one portion—the bottom piece—tied up by each pile. In the vertical method all the sides of the form for the pile must remain in place until the pile is strong enough to stand alone. On a large installation quite a sum of money can thus be saved on forms, and, naturally, the horizontal method is the more popular.

One of the best known types of this class, and among the first to be invented and practically applied, is a straight pile of square cross-section with the corners slightly chamfered. It is reinforced with $\frac{3}{4}$-in. rods, one near each corner of the square, and connected with each other at intervals of 12 in. by diagonal and straight wire ties, the lower end of the pile being protected with a pyramidal metal shoe firmly attached to the reinforcement. Provision for jetting is made, in the lower end only, by the insertion of a piece of wrought-iron pipe, about 3 ft. long bent in such a manner as to extend from the center of the point to the outside of the pile just above the shoe, where an outside connection can be made and then removed, thus making it possible to save a considerable length of pipe and at the same time make a more solid pile. This type has been used in all kinds of foundation work, both in the United States and abroad, particularly in structures along water fronts, bridges, buildings, etc.

Another well-known type of the moulded pile is tapering in shape, 16 in. in diameter at the butt and 11 in. at the point. It is octagonal in cross-section, each of the faces of the octagon containing a semi-circular longitudinal corrugation for almost its entire length. It is reinforced throughout with Clinton woven-wire cloth, and a central opening, for jetting purposes, is moulded in the concrete by the use of a tapering collapsible mould, thus obtaining an opening for the water jet at small expense. After the pile has been placed, this open-

ing can be filled with grout and fine concrete, thus making a solid pile. A driving-head, somewhat similar to that previously described, is used with this pile when it is driven into place by impact. It is claimed for this pile that, as the corrugations increase its superficial area, its bearing capacity is increased correspondingly, and the corrugations assist materially in the escape of the water, making it more easily put in position.

A third type, circular in cross-section, is formed in an entirely different manner, in that it is not moulded or cast but is rolled mechanically into shape. A sheet of ordinary black wire fabric, of No. 16 gauge, with ½-in. mesh, and of the length required to make the pile, is attached to the rod which serves as the winding core and later as a part of the reinforcement of the pile, by small rivets driven into holes slightly smaller. The other edge of the sheet is attached to a movable platform which is pulled toward the core as it is wound up. On this sheet is placed the other reinforcement, and the whole is covered with a layer of fine concrete. The mass is then wound up like a jelly roll, and fastened with wire ties every 6 in., and more frequently where needed. In some of the later piles the method of fastening the outer edge of the wire cloth has been changed by omitting the wire ties and substituting a rivet with its end upset so that it is slightly larger in diameter than the mesh of the cloth. These rivets are inserted in the outer edge of the cloth with this upset end upward, and, as the pile is rolled up, they are forced into the meshes of the fabric in the body of the pile, thus tying or rather buttoning it together. The platform on which the pile is moulded is inclined, so that, without any lifting, the completed pile is rolled therefrom to the car which is to carry it to the seasoning yard. The platform of the car is also inclined, and, when the car has reached the yard, the pile is rolled to its place on skids. By this method no large plant, except the rolling device, is required to handle the pile during its formative stage. When these piles are to be driven into place, a driving head is used, and when jetted, a wrought-iron jetting pipe forms the winding core. Piles of this kind, 61 ft. long, have been driven, pulled out for examination, found to be intact, and re-driven. Like other piles of this class, they have stood very severe treatment. In one instance an attempt was made to destroy a pile by over-driving; later the pile was pulled up and the only apparent injury was the

FIG. 1.—TYPE B. MADE-IN-PLACE CLASS: FOUNDATION READY FOR CAPPING.

FIG. 2.—TYPE 3. MOULDED CLASS: EXAMPLE OF PILE DRIVEN TO REFUSAL SEVERAL TIMES.

mashing or brooming of the upper 2 ft. This type has been used in the foundations of some of the stations of the Brighton Beach Railroad, in Brooklyn, N. Y., in filled ground. It has also been used in other places throughout the United States.

At Kansas City, Mo., solid octagonal piles (unpatented) were used in the foundations of the Sixth Street Viaduct. These piles were moulded horizontally, and, when given sufficient time to harden, were successfully placed. The contract was let in the fall, and as it was found to be impossible to mould enough piles to complete the work, it became necessary to use some creosoted piles.

Moulded concrete piles were used during 1908 in the construction of a boardwalk, 50 ft. wide, at Long Beach, Long Island. These piles were moulded horizontally, and were 11 in. square at the butt and 8 in. square at the point, with the edges chamfered; they were reinforced with a 1-in. bar in the center. The piles were 25 ft. long and were jetted 15 ft. into the sand by the use of a twin outside jetting apparatus which worked very successfully.

In the construction of a pier at Atlantic City, a few years ago, there was used a moulded concrete pile with a bulbous end of much larger diameter than the body of the pile. These piles were moulded on end and jetted into place. The pile was 25 in. in diameter, and the bulb at its greatest diameter was 3 ft. 6 in. This pile was reinforced with eight $\frac{3}{4}$-in. bars symmetrically placed around a $2\frac{1}{2}$-in. jet pipe in the center.

During 1908 a moulded pile of the same general style was used in the construction of a boardwalk at Atlantic City, N. J.

In 1906 the Atlantic and Birmingham Railroad built two piers at Brunswick, Ga., on moulded piles somewhat like the pile of this class which was described first. The piles were 16 in. square, and were straight, except that the lower 10 ft. tapered to 8 in. square at the point, and the edges were chamfered. The top of each pile, for 22 in., was 10 by 16 in. to allow of a bolted connection with the wooden girders of the superstructure. They were reinforced with four $1\frac{1}{4}$-in. rods held in position at 12-in. intervals with wire clips, and fastened to the jet pipe at the point. These rods were extended to within 3 ft. of the top of the pile, the reinforcement in this part being woven-wire fabric. About 1 500 of these piles, from 32 to 50 ft. long, were installed. One would naturally expect that timber piles would have

been used, as Brunswick is the shipping port for yellow pine and its products, but it was not practicable to use timber on account of the marine borers. The supports of the timber docks now in use require constant repairs, and the timber pile does not last long enough to pay for its installation, even though it may be very cheap. On the other hand, these new concrete piles make a permanent pier and will never require replacing.

In a power-house foundation at Bridgeport, Conn., a composite timber and concrete pile was used. The site was on the banks of a tidal stream where the muck deposit was very deep, and this pile was used in order to save the expense of excavation and concrete footings. The timber pile was driven about flush with the surface of the ground, then a hollow, steel pipe follower, 12 in. in diameter, was used to drive it below water level. The follower was then filled with concrete and withdrawn, leaving the concrete in place on the butt of the pile. A mechanical bond between the timber and the concrete was obtained by the use of long metal dowels which were driven into the butt of the timber pile before the follower was placed in position.

Moulded concrete piles, of rectangular cross-section, and having a male and female joint on two edges, have been used as sheet-piles in many cases during the past few years. One type is 6 by 14 in. in cross-section, with a reinforcing rod near each corner, the rods being held in position by wire clips at regular intervals. These sheet-piles are installed in a manner similar to that heretofore described, either by jetting or driving, or a combination of both, as may be desirable.

There are in use to-day many other varieties of concrete piles which differ in some respect from those described, either in the reinforcing, the shape, or the method of installation, but their general characteristics will place them in one of these classes.

The different shapes and sections of the various concrete piles have led to considerable discussion between their respective advocates as to the relative bearing power of each pile, most of the contention referring to the shape of the pile and whether the tapering or the straight shape (in which the pile is of uniform cross-section throughout) has the greater supporting value.

The tapering pile depends for its bearing power more on its wedge shape and the skin friction thus developed than on its efficiency as a column, while the straight pile relies on the skin friction developed

FIG. 1.—TYPE 3, MOULDED CLASS: MACHINE FOR ROLLING THE PILE.

FIG. 2. TYPE 3, MOULDED CLASS: PILE 14 IN. DIAMETER; 50 FT. LONG; WEIGHT, 3½ TONS HAULED 4 MILES TO SITE OF WORK.

by its exterior surface, and also on its strength as a column. The amount of skin friction developed by any pile depends, not only on the character of its surface, but also on the area of that surface. Considering for the present only the area, and assuming two piles of equal length and having butts of the same size, one tapering to 6 in. at the point while the other is straight throughout, it will be found that the straight pile has a larger surface area on which to rely for skin friction, and has a larger bearing surface at the point. If these piles are each 14 in. at the butt and 20 ft. long, the tapering pile will have a surface area of 52.36 sq. ft. as compared with 73.30 sq. ft. for the straight pile; while the point area of the tapering pile will be 0.196 sq. ft. and that of the straight pile 1.069 sq. ft., the latter having almost 5½ times as much area available to transmit its load as a column, about 1.4 times as much superficial area, and 1.7 times as much volume, which also represents its displacement.

The relative compressions may be seen by Table 1.

TABLE 1.

Type of Pile.	Diameter, in Inches.		Circumference, in Feet.		Cross-Section Area, in Square Feet.		Surface Area, in Square Feet.	Volume, in Cubic Feet.
	Butt.	Point.	Butt.	Point.	Butt.	Point.		
Tapering.	14	6	3.665	1.571	1.069	0.196	52.36	12.64
Straight..	14	14	3.665	3.665	1.069	1.069	73.30	21.42

Comparing the two piles again, for the purpose of ascertaining the relative frictional values of their upper and lower parts, it will be found that the tapering pile has a much larger proportion of its surface area in the upper than in the lower zone, which indicates that it must depend on the upper layers of the ground for the greater part of its supporting power, while the straight pile has its surface area uniformly distributed and can utilize all the strata through which it passes.

It is apparent from Table 2 that 32.5% of the surface of the tapering pile is in its upper quarter while only 17.5% is in the lower quarter. The Building Department of New York City allows 15 tons per sq. ft. on rock and 10 tons per sq. ft. on firm gravel or clay, when foundations

are carried down through earth by piers of stone, brick, or concrete; on this basis, assuming a load of 20 tons on each pile, and that each has been driven to the firm gravel, so that it can act as a column, the tapering pile, with its 0.196 sq. ft. of area at the point can transmit as a column only 1.96 tons, leaving the remainder of the load, 18.04 tons, to be supported by skin-friction, and about one-third of this would have to be supported by the upper stratum of the ground. The straight pile, having 1.069 sq. ft. of area at the point, can sustain as a column 10.69 tons, leaving 9.31 tons to be carried by skin friction, and about one-quarter of this, or $2\frac{1}{3}$ tons, would be supported by the upper stratum of the ground.

TABLE 2.

	Circumferences, in Feet.		Superficial Areas of Zones, in Square Feet.		Percentage of Zone Areas to Entire Pile.	
	Tapering	Straight.	Tapering.	Straight.	Tapering.	Straight.
Butt	3.665	3.665	17.020	18.325	32.5%	25%
One-quarter	3.142	3.665	14.400	18.325	27.5%	25%
One-half	2.618	3.665	11.780	18.325	22.5%	25%
Three-quarters	2.094	3.665	9.160	18.325	17.5%	25%
Point	1.571	3.665				
			52.36	73.30	100.0%	100%

The basic reason for the use of piles for foundations is that the surface ground at the site has not sufficient stability to support the load; therefore piles are used as a means by which to transmit it to a lower and more substantial stratum. Structures supported on tapering piles, in districts subject to floods and the scour of currents, are likely to settle seriously if erosion takes place, and, under these conditions, the piles should be driven to a firm bearing stratum and not rely on the upper one to support the load.

The supporting power of a pile due to frictional resistance depends largely on whether the surface of the pile is smooth or rough, thus a rough-surfaced concrete pile will have more sustaining power, due to that element, than a pile of the same size with a smooth metallic or wooden surface. The supporting power due to the same factor

also depends on the material in which it is driven, which may be anything from a fluid muck to a solid gravel or clay; it is possible to have any of these features in combination with those above mentioned, therefore it is important to study carefully all the conditions of each problem before deciding which type of pile to adopt.

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PAPERS AND DISCUSSIONS

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COPYRIGHT IN DRAWINGS OF A TECHNICAL CHARACTER.*

By D. A. Usina, Assoc. Am. Soc. C. E.

The copyright law enacted March 4th, 1909, to go into effect July 1st, 1909, contains provisions which may be of great importance to engineers, in that it provides specifically for copyright on "Drawings (or plastic works) of a scientific or technical character," and in that the securing of the copyright is made an extremely simple matter.

Prior to the passing of this law there has been no certain protection against the pirating of designs for engineering structures. Where a design though new has involved only the skill of the engineer, it has been unpatentable, and unless the work could be classed under the fine arts there has been no reasonable hope of protection by copyright. In many cases designs involving a high degree of engineering skill and gotten up at great expense have been copied by others with no moral justification whatever. National, State and city governments and private parties have asked for bids to be accompanied by original designs and have appropriated the design of one bidder and let the contract to another—a species of piracy distinguished legally but not morally from the piracy of works of literature or the fine arts.

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

It is hoped that the new law will put a material check on such practices by reserving to the proprietor of such drawings the sole right of printing or copying them. It is doubted if the law can be invoked to prevent the building of the structures shown in such drawings, with the possible exception of works which might be classed in the fine arts. The limitations which can be put upon the use of the drawings, however, should afford a substantial degree of protection, and the ease with which copyright can be secured makes it undoubtedly worth trying for whatever protection can be secured.

The language of the Act relative to technical drawings is as follows:

"That the application for registration shall specify to which of the following classes the work in which copyright is claimed belongs: * * * (*i*) Drawings or plastic works of a scientific or technical character."

Taken in connection with the remainder of the law, there is little or no doubt that this language was intended to render such drawings copyrightable. The view of the Register of Copyrights is:

"that the language of the Act would authorize the deposit and registration in this office for copyright protection of drawings of a scientific or technical character such as under the present law would not be included in the term 'drawing,' which seems to be confined to artistic drawings. Examples of such works would be drawings for machinery, for engineering construction, for architectural works, etc. While these drawings have to some extent been registered in the Copyright Office heretofore, it has always been a question whether the law authorized the registration and whether copyright protection would follow."

Protection is extended to "all the copyrightable component parts of the work," such for example as the drawings relating to any complete and mentally segregable part of the whole structure.

Copyright is secured by publication with the required notice affixed to each copy; or, where "not reproduced for sale," by depositing a photograph or other reproduction with the Register of Copyrights. But engineering drawings are usually offered for sale (as part of the bidder's work), and, in such cases, the copyright is secured by affixing a notice of copyright; and after thus securing the copyright there must be promptly deposited in the Copyright Office two complete copies.

Several permissible forms of notice are provided. It is sufficient if the word "Copyright" be applied to each sheet of drawings, together

with the name of the proprietor and the date. It is customary now to apply the name and date to drawings; and the addition of the word "Copyright" is the simplest matter in the world. For what it is worth (and apparently it will sometimes be worth much) the word "Copyright" should be added to all original drawings and copies thereof.

In the case of drawings of especial value it is advisable also to deposit copies promptly with the Register of Copyrights and to secure a certificate of such deposit, so as to obtain the apparent sanction of the Government, and to indicate such fact on the face of the drawings. For drawings of less value the depositing of copies may be delayed until it is thought necessary to threaten or bring suit for infringement. The securing of copyright is probably effected by the mere notice. The deposit of copies is an additional formality which, though required to be made "promptly," and though a necessary preliminary step before a suit for infringement can be brought, does not seem necessary to the securing of the right.

Copyrighted works may be sold or furnished to others with any reasonable limitation upon their use. For example, a bidder may furnish an original design with a notice of copyright and a special notice that the copyright and the copies submitted constitute parts of the bid and remain the property of the bidder in case his bid is not accepted, and that the design is not to be copied or submitted for bids from others.

An interesting feature of the new law is the apparent sanctioning of actions against the United States and its officers; but much weight cannot be placed on this construction, in view of the broad prohibition in the general law against actions "sounding in tort" against the United States. Congress has provided no remedy for wrongs committed by the National Government, except for violations of contract, express or implied. The provision of the new law bearing on this subject relates specifically to the recovery of costs in suits for infringement and states that in such suits,

"except when brought by or against the U. S. or any officer thereof, full costs shall be allowed, and the court may award to the prevailing party a reasonable attorney's fee."

It is unnecessary to consider the question here at length, but it may be stated generally that after a tort is committed by the United States there is no adequate remedy, but an injunction may be secured

against a repetition of it; and perhaps, if one has sufficient notice of an intention to infringe the copyright, such infringement can be enjoined in the first instance. Possibly the framers of the law had such injunctive proceedings in view when they framed the above section exempting the United States and its officers from the payment of costs.

The Act provides also that "any person who willfully and for profit" infringes a copyright may be punished by a fine and imprisonment; and an infringer who had received such a special notice as is above referred to would be infringing willfully and for profit, and would hesitate to run the danger of fine and imprisonment.

Under the old copyright law there were only a few cases in which it was attempted to secure a copyright upon a purely technical drawing. It was necessary to enter the drawings in the office of the Librarian of Congress, and the Register of Copyrights permitted such entry only under protest and was of the opinion that such drawings were not copyrightable. The validity of the copyright was generally doubted. The new law, which specifies such works particularly as one of the classes in which copyright may be claimed, and permits the securing of the copyright by applying to the drawings a notice that copyright is claimed, is a material advance toward the protection of a kind of intellectual property of far greater dignity and usefulness than circus posters and similar matters, to which the previous law had extended protection. To quote the language of the Constitution upon which is based the power of Congress to make a copyright law, technical drawings do more "to promote the progress of science and useful arts" than many of the classes of work heretofore provided for.

The practical value of the new law to engineers cannot be predicted. We can only hope. It cannot be used to protect functional equivalents of the thing represented by the drawings. That is the purpose of the patent law, and protection thereunder runs for seventeen years only, while copyrights may last fifty-six years. It is quite possible for the courts to construe the new law so narrowly as to destroy its value. But it will probably be construed so that it will protect the engineer against the copying by others of the substance or of any material part of his original designs.

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PAPERS AND DISCUSSIONS

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HYDRO-ELECTRIC POWER IN CANADA.

By Cecil B. Smith, M. Am. Soc. C. E.

To be Presented September 1st, 1909.

Introductory.

To view justly the progress which has been made in Canada, in developing water-powers, up to the present time, certain premises must be established, climatic conditions realized, the trend of population, habits, and occupations of the people pointed out, and, above all, the peculiarly favorable topography of the country described.

The population of Canada is familiarly spoken of as agricultural in character, but this is only relatively true, as large and important industries flourish throughout the Eastern Provinces, and, indeed, it is painfully evident that even in this young and sparsely populated country the increase in population is almost entirely urban in character, except on the great wheat plains of the Middle West.

Industries are based on labor, raw materials, and power, and only in the last named would the ultimate conditions be unfavorable to continued progress if coal were the only source of power to be relied on, because Canada is very peculiarly situated as regards its coal areas.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Four areas are at present known, as follows:

The Nova Scotian, which is on the extreme Atlantic seaboard, and produces a good bituminous coal, available as far westward as Montreal;

The Souris, which is in Southwestern Manitoba, and produces a low-grade lignite of limited area and value;

The Alberta, which covers a very large area extending from the United States boundary to north of Edmonton and from the plains well up into the Rocky Mountains, the quality ranging from lignite to anthracite, and, as regards both quantity and quality, probably one of the most valuable coal deposits in the world; and

The Vancouver, which is very extensively distributed over the island and up some of the mainland valleys, chiefly developed for export to the Pacific Coast cities of the United States.

Thus it appears that coal is available on the extreme west and east, but for 2 000 miles, covering much of the most populous and fertile center of the country, it is necessary to import coal from the United States for heat and power.

From a hydraulic point of view, however, the country has been peculiarly favored, as a glance at the map will show.

The Hudson Sea is deep set into the heart of the country, and the height of land between the waters flowing to this sea and to the St. Lawrence drainage system is from 1 200 to 1 500 miles long, and, which is of more importance, it is only from 1 000 to 2 000 ft. above the sea, is not of a mountainous character, but is a perfect network of lakes, swamps, rock, and forest, and affords a good reservoir and regulator for hundreds of large rivers, which, as they reach the parent St. Lawrence, descend over precipitous Laurentian rapids, admirable as sources of hydraulic power. This condition obtains all over Canada as far west as Eastern Manitoba, and, as the rainfall averages from 25 to 40 in. per year, with no marked tendency to drought, and as the long winters store up so much moisture that the low-water period often merges into the period of autumn rains, the pinch of low water is not in many cases a serious menace, except in March of severe winters. On the great western plains the topography and climate both militate against hydraulic values, as will be dealt with later.

In British Columbia there are two diverse regions: the eastern and central portions have a low rainfall, and the glaciers feed the

rivers throughout the summer, so that the low-water period is in the late summer and autumn.

Along the Pacific Coast there is an excessive rainfall of from 100 to 150 in. per year, but it is distributed unequally throughout the seasons, and only by making use of storage lakes can extreme low-water conditions be avoided.

Until twenty years ago, hydraulic power often formed the basis of growing industries and determined frequently the location of the centers of population, but the larger centers of trade were developed by rail and water transportation and in many cases were quite distant from such sources of power.

With the advent of electric power transmission, however, these cities offered a favorable market for the sale of light and power from distant sources, and private companies, and in some cases municipalities, have invested large sums in hydro-electric constructions and transmission systems to serve the adjoining districts.

While much has been accomplished, much remains to be done, and the discussion of present accomplishments which is herein given is intended to mark a mile-stone on the road.

Of the many great rivers of Canada, the Mackenzie, Nelson, and St. Lawrence are by far the greatest, having drainage areas of 660 000, 370 000, and 565 000 sq. miles, respectively; but, outside of these, there are three districts worthy of notice from a hydraulic standpoint, namely, the Atlantic Coast, the Pacific Coast, and that great array of rivers which, in addition to the Nelson, flow into the Hudson Sea.

Atlantic Coast.

On the Atlantic seaboard the configuration of the Provinces of Prince Edward Island and Nova Scotia does not provide large drainage areas, and, also, in the presence of extensive coal measures and coal mining, the incentive to hydro-electric development has been lacking; indeed, a recent installation of steam-electric equipment having a capacity of 500 kw. at the pit mouth, from which electric energy is transmitted at 11 000 volts to Amherst, N. S. (distant 6.5 miles), may be but the pioneer of much more extensive installations of a similar character in other parts of America.

Farther westward in New Brunswick, there exists the condition more usual in Canada, namely, rivers having large drainage areas,

PACIFIC OCEAN
ARCTIC OCEAN
ATLANTIC OCEAN
VICTORIA ISLAND
BAFFIN ISLAND
FOX CHANNEL
HUDSON STRAIT
SOUTHAMPTON ISLAND
HUDSON BAY
JAMES BAY
LABRADOR
NORTHWEST TERRITORIES
YUKON
BRITISH COLUMBIA
ALBERTA
SASKATCHEWAN
MANITOBA
ONTARIO
QUEBEC
NEW BRUNSWICK
NOVA SCOTIA
NEWFOUNDLAND
PRINCE EDWARD I.
CAPE BRETON I.
ANTICOSTI I.
GULF OF ST. LAWRENCE
UNITED STATES
Dawson
Mackenzie R.
Great Bear Lake
Great Slave Lake
Dubaunt L.
Peace R.
L. Athabaska
Athabaska R.
Reindeer L.
Churchill R.
Nelson R.
Hayes R.
Severn R.
Winnipeg R.
Lake Winnipeg
Winnipegosis
Manitoba
Edmonton
Calgary
Regina
Prince Rupert
Vancouver
Victoria
Seattle
Portland
Columbia R.
Fraser R.
Kootenay L.
Crow Nest Pass
Belly R.
Saskatchewan R.
Red R.
Rainy L.
Lake of the Woods
Fort Frances
Fort William
Port Arthur
L. Superior
Sault Ste. Marie
Turbine
Sudbury
Cobalt
Georgian Bay
L. Huron
L. Michigan
Orillia
Peterborough
Toronto
Hamilton
London
L. Ontario
Niagara Falls
Buffalo
L. Erie
New York
Ottawa R.
Shawinigan Falls
Three Rivers
Montreal
Quebec
Lachine Rapids
St. Lawrence R.
Chaudière Falls
Lake St. John
Abitibi
Saguenay R.
Rupert R.
Moose R.
Albany R.
Ekwan R.
Attawapiskat R.
Nottaway R.
Grand Falls
Woodstock
Fredericton
St. John
Moncton
Amherst
Halifax
Sydney
Labrador
Leaf R.
Eskimo R.
Hamilton R.
Churchill
Wabush L.
Lake Champlain
Burlington
Rutland
Montpelier
Concord
Boston
Portland
Bangor
Augusta
Lewiston
Rochester
Albany
Syracuse
Utica
Buffalo
Cleveland
Detroit
Toledo
Chicago
Milwaukee
Minneapolis
Duluth
Fargo
Grand Forks
Helena
Spokane
Tacoma
Seattle
Portland
San Francisco
Los Angeles
Salt Lake City
Denver
Omaha
Kansas City
St. Louis
Memphis
New Orleans
Galveston
Houston
Dallas
Fort Worth
Oklahoma City
Wichita
Topeka
St. Paul
Des Moines
Davenport
Peoria
Indianapolis
Columbus
Pittsburgh
Philadelphia
Baltimore
Washington
Richmond
Norfolk
Charleston
Savannah
Jacksonville
Tampa
Miami
Key West
Atlanta
Birmingham
Nashville
Louisville
Cincinnati
Knoxville
Chattanooga
Little Rock
Shreveport
Beaumont
Corpus Christi
El Paso
Albuquerque
Santa Fe
Phoenix
Tucson
Yuma
San Diego
Reno
Carson City
Boise
Pocatello
Salt Lake City
Cheyenne
Omaha
Sioux City
Sioux Falls
Aberdeen
Bismarck
Minot
Williston
Glasgow
Havre
Great Falls
Butte
Missoula
Kalispell
Spokane
Walla Walla
Yakima
Seattle
Tacoma
Olympia
Aberdeen
Astoria
Portland
Eugene
Salem
Medford
Klamath Falls
Crater Lake
Mount Rainier
Mount Hood
Mount Shasta
Lassen Peak
Yosemite
Sequoia
Death Valley
Mojave Desert
Colorado R.
Grand Canyon
Rio Grande
Pecos R.
Brazos R.
Trinity R.
Red R.
Arkansas R.
Missouri R.
Mississippi R.
Ohio R.
Tennessee R.
Cumberland R.
Potomac R.
Susquehanna R.
Delaware R.
Hudson R.
Connecticut R.
Merrimack R.
Kennebec R.
Androscoggin R.
Penobscot R.
St. John R.
Gulf of Mexico
Gulf of California
Gulf of Alaska
Bering Sea
Aleutian Islands
Hawaiian Islands
Honolulu
Pearl Harbor
Midway Islands
Wake Island
Guam
Philippine Islands
Manila
Luzon
Mindanao
Samoa
Fiji Islands
Tonga
New Zealand
Auckland
Wellington
Christchurch
Dunedin
Sydney
Melbourne
Brisbane
Perth
Adelaide
Darwin
Tasmania
Hobart
Launceston
Port Moresby
New Guinea
Solomon Islands
New Caledonia
Noumea
Tahiti
Papeete
Marquesas Islands
Easter Island
Galapagos Islands
Panama Canal
Panama City
Colón
Cristóbal
Havana
Cuba
Jamaica
Kingston
Haiti
Port-au-Prince
Dominican Republic
Santo Domingo
Puerto Rico
San Juan
Virgin Islands
Bermuda
Hamilton
Nassau
Bahamas
Caribbean Sea
Gulf of Mexico
Mexico
Mexico City
Guadalajara
Monterrey
Veracruz
Acapulco
Tampico
Mazatlán
Hermosillo
Chihuahua
Durango
Zacatecas
San Luis Potosí
Guanajuato
Querétaro
Puebla
Oaxaca
Tehuantepec
Yucatán
Mérida
Campeche
Belize
Guatemala
Guatemala City
Honduras
Tegucigalpa
El Salvador
San Salvador
Nicaragua
Managua
Costa Rica
San José
Panama
Colombia
Bogotá
Medellín
Cali
Barranquilla
Cartagena
Maracaibo
Caracas
Venezuela
Trinidad
Port of Spain
Tobago
Guyana
Georgetown
Suriname
Paramaribo
French Guiana
Cayenne
Brazil
Manaus
Belém
Fortaleza
Recife
Salvador
Rio de Janeiro
São Paulo
Santos
Brasília
Belo Horizonte
Porto Alegre
Curitiba
Florianópolis
Iguaçu Falls
Foz do Iguaçu
Paraguay
Asunción
Argentina
Buenos Aires
La Plata
Rosario
Córdoba
Mendoza
San Juan
Tucumán
Salta
Jujuy
Bariloche
Patagonia
Tierra del Fuego
Ushuaia
Strait of Magellan
Punta Arenas
Cape Horn
Chile
Santiago
Valparaíso
Viña del Mar
Concepción
Temuco
Valdivia
Puerto Montt
Chiloé Island
Atacama Desert
Antofagasta
Iquique
Arica
Peru
Lima
Callao
Cuzco
Machu Picchu
Lake Titicaca
Puno
Arequipa
Nazca
Ecuador
Quito
Guayaquil
Galapagos Islands
Bolivia
La Paz
Sucre
Santa Cruz
Cochabamba
Oruro
Potosí
Tarija
Uyuni
Salar de Uyuni
Lake Titicaca
Iceland
Reykjavík
Greenland
Nuuk
Ilulissat
Qaanaaq
Thule
Baffin Island
Iqaluit
Resolute
Cambridge Bay
Yellowknife
Inuvik
Whitehorse
Dawson
Fairbanks
Anchorage
Juneau
Ketchikan
Sitka
Prince Rupert
Kitimat
Terrace
Prince George
Kamloops
Kelowna
Penticton
Vernon
Nelson
Trail
Castlegar
Cranbrook
Fernie
Crowsnest Pass
Lethbridge
Medicine Hat
Red Deer
Drumheller
Banff
Jasper
Canmore
Kananaskis
Canmore
Revelstoke
Golden
Field
Yoho National Park
Glacier National Park
Mount Robson
Mount Assiniboine
Mount Logan
Mount Saint Elias
Kluane National Park
Wrangell–Saint Elias National Park
Glacier Bay National Park
Tongass National Forest
Chugach National Forest
Denali National Park
Katmai National Park
Lake Clark National Park
Kenai Fjords National Park
Gates of the Arctic National Park
Arctic National Wildlife Refuge
Prudhoe Bay
Barrow
Point Hope
Kotzebue
Nome
Bering Strait
Wrangel Island
Chukchi Sea
Beaufort Sea
Amundsen Gulf
Coronation Gulf
Victoria Island
Banks Island
Melville Island
Prince of Wales Island
Somerset Island
Boothia Peninsula
King William Island
Gjoa Haven
Taloyoak
Rankin Inlet
Coral Harbour
Southampton Island
Repulse Bay
Igloolik
Pond Inlet
Arctic Bay
Grise Fiord
Ellesmere Island
Alert
Eureka
Nunavut
Nunavik
Nunatsiavut
Torngat Mountains
Torngat Mountains National Park
Labrador City
Wabush
Churchill Falls
Goose Bay
Happy Valley-Goose Bay
Cartwright
Battle Harbour
Red Bay
L'Anse aux Meadows
Gros Morne National Park
Corner Brook
Stephenville
Port aux Basques
Gander
Grand Falls-Windsor
Clarenville
Bonavista
Twillingate
Fogo Island
Trinity
Terra Nova National Park
Cape Spear
St. John's
Conception Bay
Placentia
Argentia
Avalon Peninsula
Burin Peninsula
Grand Bank
Fortune
Saint-Pierre and Miquelon
Sable Island
Cape Breton Highlands National Park
Bras d'Or Lake
Sydney
Glace Bay
New Waterford
Louisbourg
Antigonish
Pictou
New Glasgow
Truro
Amherst
Moncton
Sackville
Shediac
Fredericton
Saint John
Saint Andrews
Campobello Island
Grand Manan
Bay of Fundy
Minas Basin
Annapolis Royal
Digby
Yarmouth
Shelburne
Lunenburg
Mahone Bay
Chester
Peggy's Cove
Halifax
Dartmouth
Bedford
Sackville
Wolfville
Kentville
Windsor
Truro
Pictou
New Glasgow
Antigonish
Port Hawkesbury
Canso
Strait of Canso
Northumberland Strait
Prince Edward Island
Charlottetown
Summerside
North Cape
Confederation Bridge
Shediac
Moncton
Sackville
Amherst
Tantramar Marsh
Chignecto Isthmus
Nova Scotia
New Brunswick
Prince Edward Island
Newfoundland and Labrador
Quebec
Ontario
Manitoba
Saskatchewan
Alberta
British Columbia
Yukon
Northwest Territories
Nunavut
Canada
United States
Alaska
Hawaii
Mexico
Central America
South America
Europe
Asia
Africa
Australia
Oceania
Antarctica
MAP OF
THE DOMINION OF CANADA
SCALE OF MILES
0 50 100 150 200 250 300
NOTE: Power Stations shown thus ⊙

and having their sources in swamp, forest, lake, and muskeg, in which snow and frost linger late in April, or even until May, and feed these rivers throughout the dry months of August and September; indeed, it is usual in Canada to find the extreme low-water period in March, before the frost has ceased to hold the moisture from flowing. The Miramichi, Restigouche, and St. John are great rivers, and extensive hydraulic constructions may be expected for grinding pulp alone, in addition to serving near-by towns and cities by transmitted electricity.

A most interesting and extensive development has been commenced at Grand Falls, N. B., by the Grand Falls Power Company, where the St. John River, with its sources in hundreds of lakes of Northern Maine, falls abruptly 115 ft. over a slate barrier.

For these hydro-electric works the type of construction which has been selected is that of vertical, 10 000-h.p., 3-phase, 25-cycle, 11 000-volt units, operating under a head of 130 ft., and the designs, under which contracts for the construction of the works are about to be let, provide for a hydraulic capacity of 80 000 h.p., with a present machinery installation of 40 000 h.p., which will generate current for ferro-manganese smelting, pulp grinding, and for transmission to the cities of Woodstock (70 miles), Fredericton (125 miles), and St. John (165 miles). An interesting feature of the design is that the generators are to be placed in the wheel-pit immediately above the turbines, with a provision for the proper ventilation of this subterranean chamber as well as careful water-proofing to ensure satisfactory electrical conditions. The tunnel tail-race will be placed entirely below the level of the tail-race water, and the draft-tubes from each unit, therefore, will be brought into the top of the tunnel from one side of the wheel-pit; this, it will be noted, is a variant on the methods adopted by the Toronto and Niagara Power Company in its works at Niagara Falls, Ont.

Pacific Coast.

On the Pacific Coast, the hydraulic conditions are very different from those in other parts of Canada, for the great mountain ranges are not good reservoirs, and, although the glaciers feed the rivers until late in the summer, yet at other seasons, owing to the light rainfalls east of the Coast Range, the rivers become very much reduced

in volume; thus the excessive floods of the Fraser River, for instance, make it unsuitable for hydraulic development for power purposes.

Near the Coast, however, the rainfall is excessive, and the existence of small lakes has facilitated the construction of several interesting developments, serving Vancouver and the adjoining territory. As instances, those of the Stave Lake Power Company and the Vancouver Power Company may be mentioned.

The works of the Stave Lake Power Company, now under construction, are 35 miles east of Vancouver, on Stave River, where a head of 90 ft. is to be utilized. The drainage area, estimated at 360 sq. miles, provides a large volume of water, owing to the excessive rainfall, which varies from 100 to 150 in. per year. The flow in the river is estimated to vary from 2 000 to 50 000 cu. ft. per sec., making the river regulation somewhat difficult.

The equipment, as at present designed, will consist of five units, each of 5 000 h.p., and it is considered that the electrical output from this installation has a good prospective market in Vancouver and Westminster to which the current is to be transmitted at an ultimate voltage of 60 000.

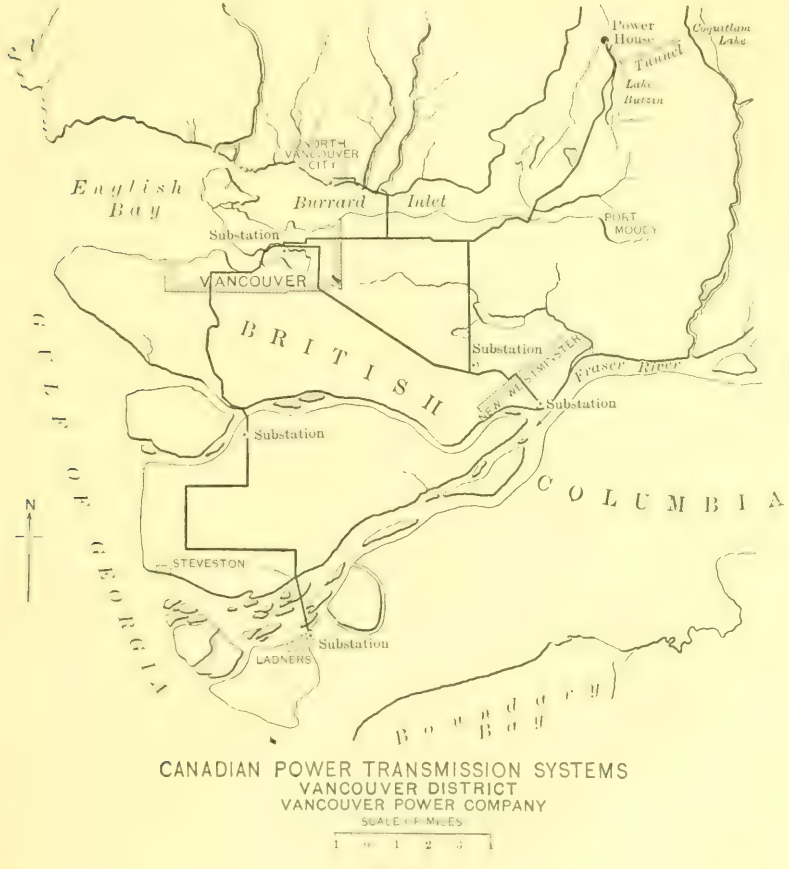
The Vancouver Power Company, Limited, has a most interesting development at Butzin Lake, 18 miles northward from Vancouver. This has been made possible by diverting the water of Coquitlam River at Lake Coquitlam into Butzin Lake by a 2½-mile tunnel having a section of 73 sq. ft.; thus the water required for operating the plant is conveyed from the Coquitlam drainage area, estimated at 100 sq. miles, the run-off varying from 300 to 5 000 cu. ft. per sec. under an excessive rainfall of 150 in. per year. At Butzin Lake the water is taken from the head-works by steel pipes to a power-house, 1 800 ft. distant, and located at sea level, 400 ft. lower.

The generating equipment is typical of the Pacific Coast, being originally composed of four units, each of 3 000 h.p.; two Pelton wheels for each unit drive a 3 000-h.p., 2 300-volt generator at 200 rev. per min., in addition to which there is also a recent installation of a 10 000-h.p. generator driven by four Pelton wheels, thus making an equipment having a total generating capacity of 22 000 h.p.

The electric current from this station is stepped up to 23 000 volts and transmitted to Vancouver, New Westminster, Burnaby, Lulu Island, and elsewhere, and is used for operating electric railways,

salmon canneries, car shops, saw-mills, and for general municipal and industrial purposes, thus serving the requirements of an urban population of about 90 000.

The transmission line consists of double towers each carrying two circuits of 3-phase, No. 2, B. & S., copper cable, and crosses navigable



CANADIAN POWER TRANSMISSION SYSTEMS
VANCOUVER DISTRICT
VANCOUVER POWER COMPANY
SCALE OF MILES
1 0 1 2 3 4

FIG. 1.

water at one place where there is a span of 2 750 ft. suspended 150 ft. above the water.

The total single 3-phase circuit length of lines is 135 miles, and the transforming capacity at present in service is 14 650 kw. at the generating station and 16 400 kw. at the various sub-stations; all

these transformers are air-cooled, and are of different sizes up to a maximum of 2 500 kw. per phase.

In the interior of British Columbia the sparse population—almost entirely south of the main line of the Canadian Pacific Railway—is engaged either in agriculture or mining, and thus it happens that, aside from small developments at Revelstoke and Nelson for municipal purposes, the only interesting hydro-electric plant in this district is that of the West Kootenay Power Company, built to serve the mines and smelters at Rossland, Trail, and in the Boundary District. It is located at Bonnington Falls, on the Kootenay River, 9 miles west of Nelson, B. C. For the new power-station—which supersedes a smaller adjoining station previously built—the design is of the vertical type, owing to the considerable variations in tail-race level, caused by gorging below the falls in high-water periods.

The hydraulic head varies from 65 to 55 ft., being least at high water, and the minimum capacity of the river is estimated at about 6 000 cu. ft. per sec., being the run-off from a mountainous drainage area of nearly 10 000 sq. miles, which, owing to the glacial sources of the river, gives a low-water period in winter and a high-water period in summer. The station is a very interesting example of complete concrete construction, including even the intake channels and draft-tubes of the turbines, and is constructed for an equipment of four units, each of 6 000 h.p., nominal rating, with the turbines of 30% overload capacity. At present, two units are installed and a third has been ordered, in addition to which the old station has an equipment of 2 700 kw., and, when required, serves Rossland and Trail over a 31-mile, cedar-pole line at 20 000 volts on two No. 0, B. & S., 3-phase circuits.

At the larger station, the triplex, high-pressure, oil-thrust pumps, etc., are developed very much along the lines of the Niagara works, and are designed to supply oil to the thrust-bearings at 250 lb. per sq. in.; the governors were built by the I. P. Morris Company, of Philadelphia, adapted and modified from the well-known Escher-Wyss type.

The step-up transforming station supplies 60 000-volt current for the new cedar-pole transmission circuits to Phœnix Mines and Grand Forks Smelters, and to Greenwood Mines, distant 79, 69, and 83 miles, respectively; and its present equipment consists of six single-phase,

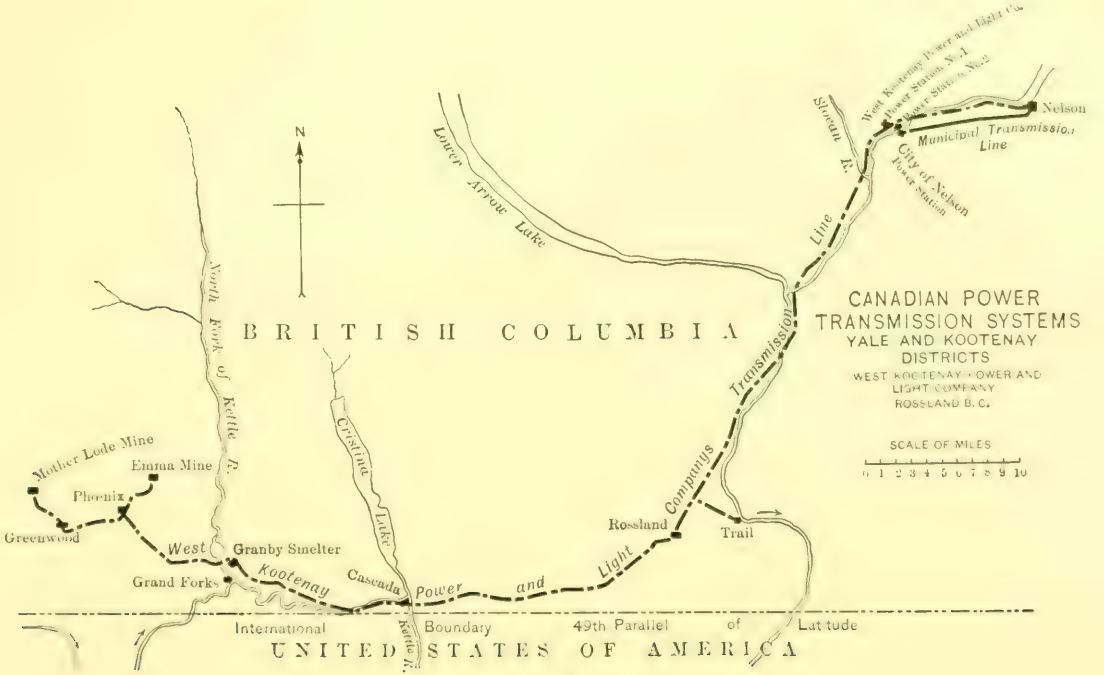

N
BRITISH COLUMBIA
UNITED STATES OF AMERICA
Lower Arrow Lake
North Fork of Kettle R.
Crystal Lake
Kettle R.
Slocan R.
Nelson
Municipal Transmission Line
City of Nelson Power Station
Transmission Line
Kootenay Power and Light Companys
Mother Lode Mine
Emma Mine
Phoenix
Greenwood
West Granby Smelter
Grand Forks
Cascade
Rossland
Trail
International Boundary 49th Parallel of Latitude
CANADIAN POWER TRANSMISSION SYSTEMS
YALE AND KOOTENAY DISTRICTS
WEST KOOTENAY POWER AND LIGHT COMPANY
ROSSLAND B.C.
SCALE OF MILES
0 1 2 3 4 5 6 7 8 9 10

FIG. 2.

water-cooled transformers, each of 1 850 kw., nominal rating, each three-phase bank being in a fire-proof compartment; there is also a bank of three single-phase, 1 250-kw. transformers held in reserve for the Rossland and Trail Districts at 20 000 volts in the event of a shut-down of the older station.

The uses for which the electric current from this station is adopted, in this district, are quite varied, consisting of mine-hauling by electric locomotives, hoisting by induction and synchronous motors, lead smelting, handling of various machinery, pumping, air compression, and general industrial and municipal purposes.

The total transforming capacity in seven sub-stations is 31 000 kw., and the active connected load is 21 000 h.p.

Immediately east of this hydro-electric development are located the great Crow's Nest Pass coal areas, but the competition is not effective, in so far as service in the mining district is concerned.

It is also anticipated that some of the adjacent mountain railways will become electrified in the near future, as the operation of rack-railways on 4½% grades has not proven satisfactory or economical.

Perhaps the most interesting feature of these works is the satisfactory maintenance of a 60 000-volt transmission line over a heavy mountainous country on a substantial but ordinary cedar-pole line. This is in sharp contradistinction to the prevailing tendency in the more level districts of Eastern Canada, where it is considered good practice to use steel towers with long spans for high-voltage transmission, but doubtless the decision was influenced by the inaccessible nature of much of the country traversed, the plentiful supply of large cedar timber in British Columbia, and also the high freight rates on structural steel, to the interior, from the Pacific Coast.

The adjoining zinc and lead smelters at Nelson, B. C., will be supplied by these works or by the municipal hydro-electric works of the City of Nelson, on the other side of the river from the large power-station just described.

This municipal power-station has now a capacity of 5 000 h.p. and an equipment of 1 000 h.p., which is about to be increased to 2 500 h.p. by the installation of a second vertical unit. The general construction has been carried out on a basis of four units having a total capacity of 5 000 h. p.

The Great Plains.

Immediately east of the Rocky Mountains, in Canada, there are two great river basins, with many subdivisions, but of these the Mackenzie River, emptying into the Arctic Sea, lies in a practically uninhabited region; however, it is interesting to note that emigration and settlement will soon change all this, and, undoubtedly, in a few generations populous communities will occupy the upper water-sheds of this river, and will require all the tools of civilization, and among them the electric current from hydro-electric stations. Flour mills are already being established on the head-waters of the Peace and Athabasca Rivers.

The Nelson River Basin.

The Nelson River drains an extensive water-shed which is a country in itself. All the inhabited portions of the Provinces of Manitoba, Saskatchewan, and Alberta, except a small area tributary to the Missouri and Mississippi Rivers, are drained by the various tributaries of this river.

This water-shed is divided into three main drainage areas, the Saskatchewan, the Red and Assiniboine, and the Winnipeg Rivers. From a hydraulic point of view, each of these rivers exhibits distinct and dissimilar characteristics.

The Saskatchewan River, of 125 000 sq. miles drainage area, rising in the eastern slope of the Rocky Mountains by many sources, fed from glaciers, and descending to the plains through many mountain gorges, offers, near its sources, reasonable opportunities for hydraulic development, and two companies are already active in the preliminary stages of contracting to supply Calgary, Alta., with electric power from hydro-electric stations to be erected on the head-waters of the Bow River. Doubtless, as population in the foot-hills increases, similar undertakings will be carried out to meet the requirements of near-by towns and cities.

Eastward of the foot-hills, however, the conditions change, and the main branches become chiefly dependent on the precipitation on the great plains, which is rather light and either evaporates or passes off in spring or autumn floods. The character of the country also changes; the streams occupy deep, wide gorges cut down into the plains, and have light grades and shifting silt and sand bottoms where dam construction would be difficult and expensive.

Speaking generally, the main valley of the Saskatchewan, eastward from the foot-hills, is not adapted to hydro-electric constructions, and, as there is coal over a large proportion of this vast area or near it, it is not probable that such constructions will ever be numerous, extensive, or profitable, except at a point on this river immediately before it enters Lake Winnipeg.

At this place, 250 miles north of the City of Winnipeg, the river passes through a series of small lakes and then, at Grand Rapids, drops quickly to the level of Lake Winnipeg. The change in level is 112 ft. in a few miles, and there is thus afforded the opportunity for an enormous development of at least 300 000 h.p. whenever the demand arises for a power development at that point.

The Red and Assiniboine branches of the Nelson River, with a drainage area of 105 000 sq. miles, are still more unfavorably located for hydraulic purposes, as the drainage basin is entirely in a plains country, where there is light rainfall and very little lake storage, thus the minimum run-off is extremely small in proportion to the areas drained, as a consequence of which, with the exception of a few quite insignificant opportunities, they may be written off the list of hydro-electric districts, not only for the foregoing reasons, but because the flood levels are usually bank full, and sand and silt bottoms, over wide flats, present expensive conditions for dam constructions. It may be remarked in passing that this drainage area is devoid of coal, except for the lignites of the Souris Valley; and the price of fuel and power is excessive, except where the coal supply is local or transmitted electricity from the eastward is possible.

The Winnipeg branch of the Nelson River drains an area of approximately 55 000 sq. miles in the State of Minnesota and in the Provinces of Ontario and Manitoba, and, from its peculiar topographical features, affords one of the most favorable locations for the development of hydraulic power in Canada, excelled perhaps only by the Niagara and the St. Lawrence and superior in some respects to the Ottawa River.

This condition is somewhat, but not greatly, offset by the moderate rainfall, which is only from 20 to 25 in. per year over the main area, but, aside from this, there is a most extensive network of lakes (of which the location, throughout the northern portion, has been, in many cases, unmapped until recently). There are thousands of lakes

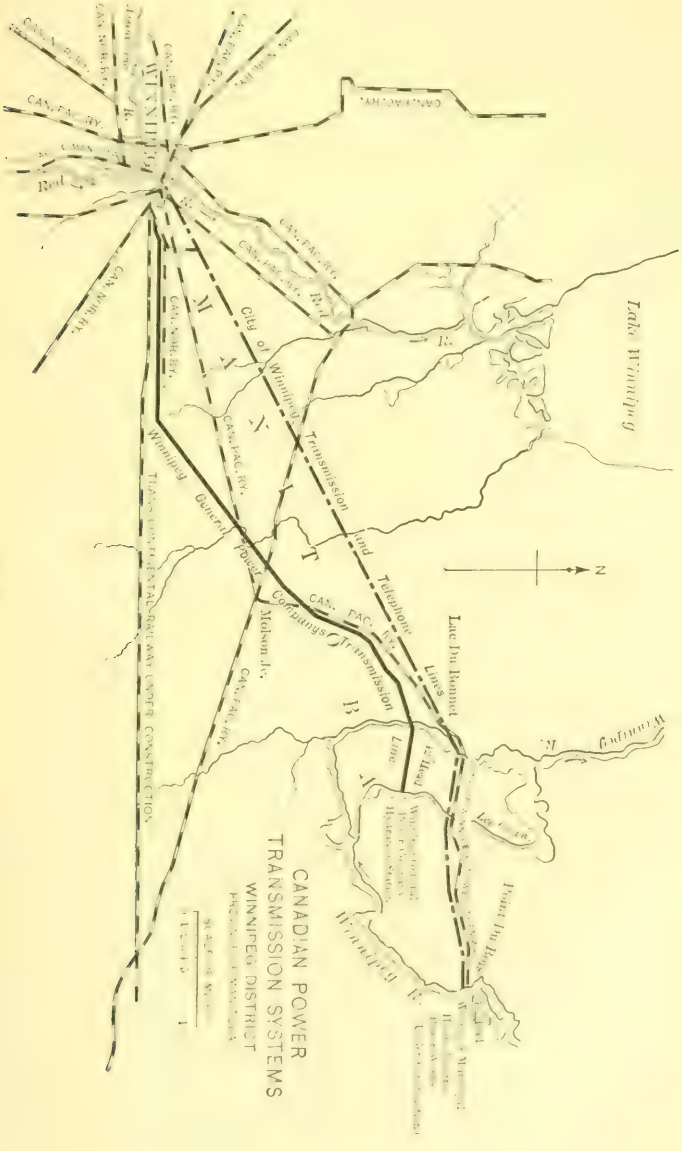

CANADIAN POWER
TRANSMISSION SYSTEMS
WINNIPEG DISTRICT
Lake Winnipeg
Lac du Bonnet
Winnipeg
Red R.
Transmission Line
Telephone Line
Shawinigan Water & Power Co.
Molson Jct.
CAN. PAC. RY.
CAN. NOR. RY.
CAN. NAT. RY.
N

Fig. 3.

of all sizes, culminating in the Lake of the Woods. There are 1 600 sq. miles of water surface on the Rainy River branch and Lac Seul on the English River branch, westward of which the two rivers unite near the eastern boundary of Manitoba and flow thence into Lake Winnipeg.

On the English River branch of the Winnipeg River no large hydraulic constructions have yet been considered seriously except near Dryden, as the country is inhabited only at a few points along the line of the Canadian Pacific Railway, which crosses the various smaller branches between the height of land and the Town of Kenora, but the Transcontinental Railway, now being constructed, will pass much nearer the more extensive water-power locations of this river.

These locations, being only the more extensive ones of the English River branch, are rated by the Hydro-Electric Power Commission of Ontario at a total of 100 000 h.p., minimum dry-weather flow, 24-hour service, and it may be considered that the commercial capacity is at least 200 000 h.p. under ordinary conditions. It seems evident that the requirements of railway operation, flour milling, pulp and paper mill work, and possibly smelting will demand a careful study of the specific value of these powers in the near future, as soon as the Transcontinental Railway is in operation.

The Seine and Rainy River branch of the Winnipeg River also possesses good hydraulic opportunities, and the Koochiching Falls, at Fort Frances, is now being developed on a basis of 14 000 h.p., for which the dam construction is now completed and power construction is under way.

The Winnipeg River proper commences at Kenora, where the river leaves the Lake of the Woods, and from this point to the mouth of the river there is a chain of falls and chutes which, as sources of economical power, is difficult to surpass or equal.

The two conditions worthy of comment are that at each power location the rocky barrier creating the fall is admirable as a foundation for a dam, thus enabling constructions to be low in first cost, and at the same time creating considerable local storage to meet daily peak-load requirements. Also, the enormous lake storages, previously referred to, very much reduce the flood discharge and increase the minimum. The latter may be taken at 13 000 cu. ft. per sec. at Kenora and 20 000 cu. ft. per sec. below the junction of the English

River, the flood discharge being not more than four times these amounts.

Table 1 is a summary of the power sites on the Winnipeg River proper, between Kenora and the mouth at Fort Alexander.

TABLE 1.—Power Sites on the Winnipeg River.

Location.	Available head, in feet.	Minimum electric horse-power; 24-hour rating.	Average electric horse power; 24-hour rating.	Remarks.
Kenora	18	21 000	42 000	5 500 h.p. installed.
Island Falls	45	56 400	112 800	
Point du Bois	46*	83 600	209 000	20 000 h.p. being installed.
Slave Falls	30*	54 500	136 200	
Upper Seven Portages	30*	54 500	136 200	18 700 h.p. installed at Lee Channel.
Lower Seven Portages	22	40 000	100 000	
McArthurs Falls	12	22 300	55 750	
Grand Bonnet	30	56 000	140 000	
Little Bonnet	10	18 500	46 200	
White Mud Falls	15	28 000	70 000	
Silver Falls	21	39 100	97 800	
Pine Portage Falls	12½	23 300	58 300	
Totals		497 200	1 204 250	

* Including raising the water level by head-works.

Of the sites mentioned in Table 1, Island Falls, Point du Bois, Slave Falls, Upper Seven Portages, and Grand Bonnet afford excellent construction opportunities. All but the first two in Table 1 are within easy transmission distance of the City of Winnipeg, and all are within short distances of two transcontinental railway systems.

The situation at Kenora illustrates the great value of water-power as applied to flour milling. The town has little else to recommend it as an industrial center, not being at a competing railway point, nor at a bulk-breaking point; neither is there any surrounding population or any fertile areas; but, in spite of these apparently unfavorable conditions, three large flour mills are located there, and use about 4 000 h.p., with a capacity of 10 000 bbl. per day. Two of these plants operate privately-owned hydraulic equipments, and the third purchases 1 000 e.h.p., at a flat rate of $10 per e.h.p.-year, from the municipal power-station. The latter has been built recently at the eastern outlet of the Lake of the Woods with dimensions to receive machinery having a capacity of 4 000 e.h.p., one-half of which has already been installed.

At the main or western outlet of the Lake of the Woods, at Kenora, there has also been built for some years, a stone dam from which water could be drawn to develop approximately 18 000 e.h.p. at low-water, but no further constructions have been commenced.

At Point du Bois the City of Winnipeg (population 120 000) has commenced the construction of a large generating station involving a 75-mile transmission line to the city, and the designs contain some interesting features. A short (1 200-ft.) head-canal, concrete overflow walls, and a rock-filled dam in the main river will raise the water 13 ft., creating a total head of 46 ft. and forming a lake 5 sq. miles in area, thus producing uniform and quiet hydraulic conditions. The dimensions of all control works, dams, etc., provide for an ultimate equipment of 60 000 h.p. The remainder of the work is to be a solid reinforced concrete structure involving an enclosed forebay room, power-house, and transformer-station equipped primarily with five 5 300-e.h.p. units, which will consist of horizontal, double-runner turbines (draft-chests only) in reinforced concrete wheel-pits and directly connected through a breast wall to 3-phase, 60-cycle, 6 600-volt, generators making 164 rev. per min.

The transformation will be to 77 000 volts, by water-cooled, oil-insulated transformers having a capacity of 3 750 kw. per phase. Similar transformation at Winnipeg will be from 72 000 volts down to 11 000 volts in a main terminal station, from which 11 000-volt feeders will distribute to three or more sub-stations for local distribution at suitable voltages; the transformers at the terminal station, however, are to be oil-circulating, and oil-cooled. The system of remote control will be fully applied at both stations.

The transmission lines, of No. 000 copper, or aluminum equivalent, will be carried on double-circuit steel towers spaced 600 ft. apart, each alternate tower being of elastic type with two legs only. All tower foundations are to be on concrete.

The city utilities are already numerous, and include Artesian water supply, pumped electrically; and the desire to be able to offer cheap power to industries, present and prospective, which would otherwise labor under a great disadvantage in a city which cannot offer bituminous coal of a good quality for less than $6.50 per ton, while anthracite coal is $10.50 per ton retail, appears to be ample justification for this ambitious project for a city of 120 000 inhabitants.

The Winnipeg Street Railway Company and its allied companies have in operation a hydro-electric station on the Lee Channel of the Winnipeg River, and a transmission line 67 miles long supplying power to a terminal station in the city.

This generating station was selected apparently without proper investigation and without an eye to the future, and the company is already expending large additional sums on the tail-race channel in order to insure sufficient hydraulic head and flow of water at low water to operate the machinery now installed, which makes a completed scheme incapable of further expansion and consisting of nine units having a combined rated capacity of 14 000 kw.

The scheme of development consists of a rock-filled dam across the Winnipeg River, which raises the normal water level 4 or 5 ft. and forces a flow of water through a 14-mile high-water channel, from which already approximately 500 000 cu. yd. of rock have been excavated.

The power-house is about 5 miles from the intake of this channel, and was designed for an operating head of 42 ft.; this, however, has not yet been realized, and it is understood that, when delivering 15 000 e.h.p. at Winnipeg, the generating station is only operating under a head of 36 ft., and is at its present limit of capacity, hence the further excavations now in progress.

The generating station has an open forebay, steel-flume wheel-pits, and units of five 2 000 kw., and four 1 000 kw., each operated by two twin horizontal turbines; the generators are for 3-phase, 60-cycle, 2 300-volt current.

The transformation is to 50 000 volts, and the transformers are oil-insulated, water-cooled, of 1 800- and 830-kw. capacity per phase, each being in a separate compartment. The total transformer capacity at the generating station is 19 500 kw. The transmission system is on double-circuit, steel towers, spaced 500 ft. apart, with protruding pieces of angle iron on each upper corner of each tower which act as lightning arresters. It is understood that this protection is very satisfactory, there having been only one shut-down during the first year's service (which commenced in 1906). The capacity of the terminal station in Winnipeg is equal to that of the transforming station at the generating works.

The utilities served consist of the city and suburban street rail-

ways, the city house-lighting, and current for the present municipally-operated street lighting. In addition to this, there is a small distribution of power for miscellaneous purposes, and two large blocks of power for flour milling. The capacity of the generating station is already fully taxed during the low-water winter season, and no effort is made to encourage large users of power by offering attractive rates; this power-station cannot be looked upon as a serious competitor to the municipal power-station now being built.

The three tributaries of the Nelson River, which have just been discussed hydraulically, converge in Lake Winnipeg and there form an enormous reservoir of 9 300 sq. miles at an elevation of 710 ft. above the Hudson Sea.

When it is considered that the drainage area feeding the outlet of Lake Winnipeg into the Nelson River proper is nearly 350 000 sq. miles, and that the water supply comes from three different and dissimilar climatic and topographical river basins, one can appreciate what a large minimum flow there is; and, as the route of the river to the Hudson Sea is rock-ribbed, in cascades, pools, and waterfalls, all that need be said here is that, should electro-chemical or other demands require very extensive water-powers, where relation to habitation is immaterial, these, to the extent of at least 4 500 000 h.p., can be readily secured on the Nelson River between Lake Winnipeg and the Hudson Sea.

Other Hudson Sea Rivers.

Of the many extensive rivers flowing into the Hudson Sea and James Bay, there is very little accurate information, and hydraulically these rivers are almost unknown except in a general way.

It can be appreciated, however, that, from the physical conformation, many enormous water-powers exist, because the table-land sources of the rivers abound in lakes at an elevation of from 600 to 1 500 ft. above the sea, and this, taken with a good rainfall and the rocky nature of the country, warrants this certain assumption. It is generally known, however, that near the sea coast most of these rivers are alluvial, and that the water-powers of value lie considerably inland, in general, at least 200 miles from the mouths.

The first development, north of the height of land which lies between the St. Lawrence River and the Hudson Sea, will probably be at Iroquois Falls, on the Abitibi River. The construction of the

Ontario Government Railway, parallel to the river, and that of the Dominion Government Transcontinental Railway east and west immediately north of Lake Abitibi, suggest the early development of Iroquois Falls for purposes of pulp grinding, or power for mines, or for general distribution to the junction city which will immediately grow up in that vicinity.

The St. Lawrence Basin.

The St. Lawrence River drainage basin is the chief seat of population in Canada, and naturally the hydro-electric developments of this region are most extensive.

The total drainage area is 565 000 sq. miles, but of this about 90 000 sq. miles comprise the water surface of the Great Lakes, which form enormous compensating reservoirs, their regulation and control, international in character, presenting a very delicate physical and political question, additionally complicated by the rights and needs of navigation; therefore it is advisable to discuss the water-powers of the main river and of its many important branches separately.

Sault Ste. Marie.—The first water-power location on the main river is at Sault Ste. Marie where Lake Superior, with 31 000 sq. miles of water surface, empties into Lake Huron, falling approximately 18 ft. and discharging about 60 000 cu. ft. per sec. at low water, thereby affording opportunity for about 90 000 e.h.p., after making allowances for various hydraulic losses. The present developments have been somewhat international in character, and have been carried out chiefly by the Lake Superior Power Company, which, with allied companies, disposes of 6 500 h.p. of hydraulic and electric power on the Canadian side of the river (out of a designed total of 20 000 h.p.) to rail mills, pulp mills, and for industrial and municipal purposes in the Canadian city.

On the United States side of the river the same company constructed a canal and power-house, under unfavorable physical conditions, designed to obtain 45 000 h.p.; but machinery for only one-half of this, or less, has ever been installed, and the chief customer, outside of municipal and small industrial users, is a carbide company having works in the vicinity; a third company, on the United States side, has a small hydro-electric development, but it is probable that no further concessions will be granted there, or, if granted,

will be under the control of the International Deep Waterways Commission, so that the needs of navigation, both as regards the maintenance of the level of Lake Superior and the water required in increased quantities for operation of the three locks at this point, may be fully conserved.

It is worthy of more than passing comment that, although these canals and locks serve an enormous commerce, their existence has been of little local value, whereas the life of the two adjacent cities of Sault Ste. Marie depends chiefly on the creation and operation of hydro-electric power, and the two Governments interested will be well advised to use every effort to facilitate rather than hinder the power use of all the water passing from Lake Superior, which is not required for locking purposes. It is evident that control works at the head of the rapids can be constructed so as to compensate for any quantity of water which may be diverted and used for power purposes, and, geographically, as an assembly point for raw materials by water and rail, Sault Ste. Marie has many advantages.

Niagara Falls.—Continuing seaward from Sault Ste. Marie, the next location which affords hydraulic opportunity is spoken of familiarly as Niagara Falls.

Lake Erie flows into Lake Ontario by a river route which, from Chippewa to Queenston, is a series of rapids and the cataract, but the cataract of Niagara Falls is barely one-half of the total difference in elevation, which is 327 ft., and that the first great hydro-electric developments have been made there with effective heads of from 136 to 175 ft. can only be looked upon, in the future, as an economic blunder, because there are potential developments which can be made at several locations in the United States and Canada using a net head as great as 300 ft., which, for as large an installation as 200 000 h.p. or greater, would not be as expensive as those now constructed, and the deduction is here offered that when industry makes the demand neither international understandings nor the makeshifts of opportunist politicians will prevent capital from carrying out the construction of these undertakings for the benefit of mankind.

This question of utilizing the hydraulic power of the Niagara escarpment is, in reality, a world problem, not a local one, because there is there latent 5 000 000 continuous electrical horse-power, considering only the low-water flow of 180 000 cu. ft. per sec. at an

CANADIAN POWER
TRANSMISSION SYSTEMS
NIAGARA DISTRICT
L A K E E R I E
L A K E O N T A R I O
UNITED STATES OF AMERICA
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Woodstock
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Stratford
Waterloo
Preston
Galt
Paris
Brantford
Hespeler
Guelph
Dundas
Hamilton
Oakville
Toronto
Grimsby
St. Catharines
Welland
Niagara Falls
Thorold
Port Dalhousie
Fort Erie
Buffalo
Niagara R.
Welland Canal
Grand R.
Thames R.
Dominion Power and Transmission Co. System
Toronto and Niagara Power Co.
Canadian Niagara Power Co.
Niagara Falls Power Co.
Ontario Power Co.
Hydro-Electric Power Commission of Ontario
Power Stations shown thus

FIG. 1.

effective head of 300 ft., and, estimating even so little as 2½ lb. of coal per horse-power-hour, this would be equivalent to a saving of 55 000 000 tons of coal per year, should this great force of Nature ever become so fully utilized; and why should it not be used in increasing amount as required? There will still be available for scenic purposes all the surplus water, which will still pass over the falls, even after the minimum has all been used for power purposes. Consider the question otherwise; this location is unique, as it is on one of the greatest highways of commerce in the world, both water-borne and by rail, and it is in the midst of a fertile and populous region, which, for 150 miles in all directions, is becoming more thickly populated year by year. The steady growth in the electric load carried by the present stations leads to the inevitable conclusion that we are only in the infancy of the utilization of this great gift of the Creator.

The level of Lake Erie can be readily controlled by compensating works at the head of the rapids, and might be designed so as to constrict the natural channel at the same rate as the artificial channels were created, either near the river or by other routes from Lake Erie to Lake Ontario, and this arrangement appears to meet and answer the only serious criticism which has tended to array the shipping interests in opposition to further utilization of the water for power purposes; and, indeed, diminution of natural flow presents certain positive advantages, as it will check that rapid erosion of the Horseshoe Falls which is now taking place.

Recurring to the actual constructions which have been made since the first station of the Niagara Falls Hydraulic Power and Manufacturing Company was completed, on the United States side of the river, in 1881, it may be briefly stated that there are six large generating stations, two on the United States side and four on the Canadian side of the river, engaged in producing electrical energy for commercial purposes, and, in addition to these, there are numerous small stations, one at Niagara Falls, Ont., and several along the route of the Welland Canal, which produce a total of probably 15 000 h.p. for miscellaneous electrical and hydraulic purposes.

It is unnecessary to describe in detail the diverse constructions, distributions, etc., which have been described so often and so minutely by various engineers, but certain broad facts will be given by which the scope of present accomplishment may be gauged.

On the United States side, the Niagara Falls Hydraulic Power and Manufacturing Company has the distinction of being first in the field, and, from diminutive beginnings in 1872, it has expanded its works until at the present time it has 77 500 e.h.p. installed, and building construction is under way for additional equipment having a capacity of 70 000 e.h.p. In addition to this, water is supplied to various industries developing 8 000 h.p. of direct hydraulic power for pulp, paper, flour, and other mills.

The power from these works is used for street railway and various industrial and electro-chemical purposes, but chiefly for the manufacture of aluminum, by the Pittsburg Reduction Company, which company has here its largest works. It is understood that this company contracts for nearly 60 000 h.p.

The head canal for this development is 110 ft. wide and 14 ft. deep at normal water level. It is constructed through the City of Niagara Falls, N. Y., for a distance of 4 400 ft., and terminates at a forebay immediately overlooking the river. The water is used under a total head of 210 ft. and a net head of about 200 ft., and is led down the side of the cliff in steel penstocks to the power-stations at the lower river level, where is installed a heterogeneous collection of direct-current and alternating-current generating units composed of various sizes from 150 to 1 000 kw. in the older station and from 3 500 to 6 500 kw. in the newer station, which facts well illustrate the growth of the art and the magnitude of the business of this station, which is entirely local, involving no transmission lines of any importance.

The Niagara Falls Power Company has two generating stations operating under an average effective head of 140 ft., one on each side of a short head-canal, and the common tail-race is a brick-lined tunnel passing to the lower river, 7 000 ft. distant.

These stations, heralded at the time as pioneers, have long since been eclipsed by the stations on the Canadian side of the river, and also suffer from certain engineering defects which, no doubt, were unforeseen at the time of construction and have since been cumulative.

The ice difficulties, after years of strenuous effort, have been largely overcome, by improvements in the intake works and by the addition of an ice shoot to the tail-race tunnel, but the first station, having a capacity of 50 000 h.p., has unprotected ice racks and is equipped with free, outward-discharge, double-runner, vertical tur-

bines. The ten generators are each 5 000-h.p., 2-phase, 2 200-volt, with revolving armatures. The second station, having a capacity of 60 000 h.p., has additional submerged-arch entrances and an enclosed forebay for further protection from ice. The turbines are of the single-runner, vertical type, with draft-tubes; the eleven generators are each 5 500-h.p., 3-phase, 2 200-volt, and are partly of revolving-field and partly of revolving-armature types. The tail-race tunnels are led from the power-stations, by very sharp curves, to a junction, and this, together with the back-piling in Power-house No. 1 due to the use of free-discharge wheels, makes it impracticable to run both stations to full capacity. It is understood that 83 000 h.p. is about all that is generated, although the market in Buffalo is being largely abandoned to the Canadian Niagara Power Company, its station being several miles nearer and belonging virtually to the same financial interests.

The power from these stations is now distributed in large blocks, chiefly to local electro-chemical consumers, and the policy of the company in creating a manufacturing district, with railway switching facilities, building sites, etc., and giving long-term power contracts, with guaranties of continuity, under penalty, has created a large and stable industrial center consuming more than 60 000 h.p. continuously from those two stations. The power transmitted to Buffalo, Tonawanda, etc., is 22 000-volt, 3-phase, 25-cycle. The pole lines are of ordinary wooden type and carry four 3-phase circuits, two on each line, some circuits being of copper and some of aluminum. Until the load was transferred to the Canadian station, all the varied electrical utilities of Buffalo, to the extent of about 40 000 h.p., were operated from these United States stations, as are still the various interurban and scenic electric railways converging at Niagara Falls.

On the Canadian side of the Niagara River four large generating stations have been constructed recently; three of these are at Niagara Falls, and the fourth is at a point 12 miles westward.

The Dominion Power and Transmission Company (formerly the Hamilton Cataract Power Company) first commenced to deliver power to Hamilton, Ont., in 1897, from a small station 12 miles westward of Niagara Falls at the foot of the Niagara escarpment, and drawing its water supply from the Welland Canal and thus from Lake Erie.

This hydro-electric development utilizes a hydraulic head of 267 ft.,

and is a very economical one, as excessive expenditures for head-works were not required. This enterprise has a progressive management, and a growing market for power has expanded it until, at the present time, the machinery equipment, including transformers, has a nominal rating of 38 400 h.p.

It is an interesting feature of this generating station that the turbines are entirely of European manufacture and design, being supplied by Italian and German makers, as the earlier experiences with manufacturers in the United States were not satisfactory.

The electric current is generated at 2 400 volts, 66⅔ cycles, and is transmitted to the surrounding towns and cities at various voltages. The chief market is in Hamilton, 32 miles westward to which three pole-line, 3-phase circuits, two of copper and one of aluminum, carry power at 40 000 volts. In addition to this, however, the company serves a network of street and interurban railways which it owns or controls, and supplies power and light in Brantford, Welland, St. Catharines, Dundas, etc., with a total of 132 miles of transmission line.

This company, owing to its favorable investment and strongly entrenched position, is able to distribute with profit at low rates for power, and has thus built up a large business very rapidly. It is understood that the peak load in Hamilton alone, which is a city of only 60 000, is considerably more than 20 000 e.h.p., which includes current for operating street and interurban railways owned by this company, and also cotton mills, knitting mills, flour mills, rolling mills, and machine shops; in fact, every phase of a busy manufacturing city is provided for.

This company also owns small hydro-electric generating stations at St. Catharines and Brantford, and may be said to control, almost exclusively, the business of supplying electricity in the Niagara District west of Niagara Falls and Thorold.

The generating station of the Canadian Niagara Power Company was constructed between 1901 and 1905. This company is practically identical with that of the Niagara Falls Power Company, and although it distributes some 2 000 to 3 000 h.p. locally to various industries in Niagara Falls, Ont., it has not entered into the long-distance transmission field, no doubt considering the local sale to be ultimately more profitable. The generating station is of the wheel-pit type, with a tail-race tunnel 2 200 ft. long, and operates under an

effective head of only 140 ft. The water is taken from the rapids immediately above the cataract, and although protecting works, submerged-arch entrances, enclosed forebay room, and sluiceways have been constructed to meet the difficult ice conditions, they are only moderately satisfactory, and a continuous winter service is maintained only by strenuous efforts and with considerable difficulty.

The generating works are complete for eleven units, each having a nominal rating of 10 000 e.h.p., and five of these machinery units are now installed.

The turbines, of Swiss design, are of the vertical type, with draft-tubes and double runners, and the generators are 12 000-volt, 3-phase, 25-cycle.

This station is inter-connected with those of the Niagara Falls Power Company by a large number of 11 000-volt underground cables, thus warranting great certainty of service; but the chief market for power is in Buffalo.

The company has a transformer station, adjoining the generating station, with an equipment of fifteen single-phase, 1 250-kw. transformers, all mounted in one room; and although these are arranged so as to be operated, if desired, at various voltages up to 60 000, the pressure used at present is 22 000.

The transmission system consists of steel poles, carrying two 3-phase circuits of aluminum cable mounted on cellulose insulators, and a third circuit is now being built. The capacity is 12 500 h.p. per circuit at 22 000 volts. At Fort Erie this transmission system has two spans of 1 600 and 2 200 ft., respectively, suspended 130 ft. above the water of the Niagara River, and it is understood that considerable difficulty was experienced in cable breakage until elastic counter-weighted suspensions were made at the end towers with suspended insulation at the center tower.

Under the franchise conditions existing between this company and the Provincial Government, and having in view the water limitations now being reported on and recommended by the International Deep Waterways Commission, it would appear to be inevitable that this company will soon be forced to enter actively into exploitation of the Canadian market, in order to enable it to increase its output sales beyond the present, which not only amounts to about 40 000 h.p. from its own station, but to this must be added some 5 000 h.p. which is

being obtained from the Electrical Development Company and exported.

It is interesting to note that the combined output of the three stations of these allied companies, *i. e.*, The Niagara Falls Power Company and the Canadian Niagara Power Company, is more than 500 000 000 kw.hr. per year, and that the load factor is more than 70%, owing to the large number of electro-chemical customers.

Between 1902 and 1906, the Ontario Power Company constructed, immediately below the cataract, a station which is equipped at present with six horizontal units, each of 10 000 or 12 000 e.h.p., generating 3-phase, 25-cycle, 12 000-volt current, at 187½ rev. per min., under a net head of 178 ft. The turbines are of German design and manufacture. The water is brought to the station by an 18-ft. steel pipe, 6 500 ft. long, which is embedded in concrete just below the hydraulic grade line, and buried in Queen Victoria Park a few feet beneath the surface.

This company selected a favorable location for its entrance works and forebay, which are located at the head of the rapids and are completed for a capacity of 180 000 h.p., and, as far as ice troubles are concerned, it may confidently expect to give a continuous service.

In several other features these works are interesting, particularly in respect to the methods of control. To handle successfully an 18-ft. column of water, 6 500 ft. long, moving at 15 ft. per sec., under ordinary commercial electrical conditions, was no mean task, but it is understood that, by means of a spiral overflow structure, this has been accomplished. The electrical control is centered in the transforming and distributing station, which is 600 ft. from the generating station and on the high bluff overlooking the cataract. At this point all the current from the generators to the high-tension lines is controlled. The transforming station is equipped with eighteen single-phase, oil-insulated, water-cooled transformers, each of 3 000 kva., each bank being in a separate compartment. This company has created several allied companies which distribute 62 500-volt and 12 000-volt current in the United States and Canada. Its extreme eastern delivery is at Syracuse, N. Y., 161 miles distant, and it distributes current westward to St. Catharines, Welland, and Port Colborne; but in view of the contract it has now entered into with the Hydro-Electric Power Commission of Ontario for the supply of from 25 000 to 100 000 h.p. at from $9.00 to $9.40 per h.p. per year for 12 000-volt continuous power,

it appears evident that its sphere of influence in Southwestern Ontario is in the near future to become very considerable, and the output of its station increased to at least 50 000 h.p.

At present its maximum load is 25 000 h.p. in the United States and 5 000 h.p. in Canada, sales being at the rate of 110 000 000 kw-hr. per year.

This company's transmission lines are very extensive and varied in character, and comprise several hundred miles of double-circuit and single-circuit, aluminum lines, chiefly mounted on steel towers carrying one circuit each. Some of the branch lines and 12 000-volt lines, however, are on timber, **A**-frame towers or cedar poles; the various constructions have been carried out substantially and scientifically, but are subject to the criticism of transmitting at only 60 000 volts for distances up to 160 miles, whereas, at the present time, 80 000-volt transmissions are in use and 100 000-volt projects are under construction; however, it is true that the suspended insulator has not yet been fully developed to unquestioned satisfaction, and, at 100 000 volts, air emanations from conductors, line troubles, etc., are certain to be considerable.

In 1903 the Toronto and Niagara Power Company was formed by parties interested in the Toronto Street Railway Company and the Toronto Electric Light Company, and, after negotiations to buy electric current from other Niagara companies had failed, a lease was secured from the Provincial Government to build a generating station in Queen Victoria Park at Niagara Falls at a site lying between those of the Canadian Niagara Power Company and the Ontario Power Company, the works of both these companies being then under construction. The location chosen was notable in two particulars, the intake being almost wholly in the surging rapids above the cataract, while the tail-race tunnel was located so as to emerge beneath the cataract itself. The construction of these works proceeded from 1903 to 1907.

Preliminary to construction, it was most difficult to construct the coffer-dams and keep them water-tight, and, together with the extensive overflow dam, etc., afterward constructed, made a heavy investment necessary. The wheel-pit is constructed for eleven units, each of 8 000 kw., and four of these units are now installed. The operating head is about 140 ft. and the station is equipped with

Francis double-runner turbines, constructed by United States manufacturers on Swiss models. In various items, however, these turbines differ from previous constructions. The draft-tubes are carried outward to twin, parallel, tail-race tunnels, and emerge from underneath so as to maintain the water seal. The penstocks are alternately right and left, in groups of two, thus economizing in wheel-pit length. The supporting structures for the vertical main shafts of the units are concrete arches instead of steel girders. Altogether, these works give evidence of advance in the art of wheel-pit construction. The power-house is a most ornate structure; indeed, this company has carried on all its operations with a lavish hand, and a resulting large investment. Its generating company is known as the Electrical Development Company, and the 3-phase, 25-cycle, 12 000-volt current generated is delivered outside of Queen Victoria Park to a transformer station whence the Toronto and Niagara Power Company transmits it at 60 000 volts to Toronto, 80 miles distant (population 300 000), over a double-circuit steel-tower line. This line is carried across Hamilton Beach and Canal and the Welland Canal by pairs of very high and heavy towers; otherwise the transmission line passes through a level cultivated country without serious transmission difficulties, although including various spans across small valleys of more than 1 000 ft. each.

The transforming stations are built apparently with a great effort to attain safety, each single-phase transformer being in a separate compartment, and there is very extensive concrete cell construction for bus-bars, lightning-arrester compartments, etc. The Toronto terminal station also has an auxiliary water-tower for supplying water to transformers in addition to the water service from the city mains. The current delivered at this station at 55 000 volts is stepped down to 11 000 volts and is then carried by underground feeders to various sub-stations in the city where it is transformed into 600-volt, D. C. and various voltages, A. C.; also, the frequency is changed from 25 to 60 cycles. Altogether, from 20 000 to 22 000 e.h.p. are used by the two subsidiary companies, in addition to sales at Niagara Falls to the Canadian Niagara Power Company for export, as before mentioned, and there are also subsidiary companies formed to distribute eastward in the United States.

This brief description of the capacities of the various Niagara

generating stations, amounting to 450 000 e.h.p. in Canada, when the stations described are fully equipped, and the fact that several other companies own charters and water rights, and are anxious to enter the field if given opportunity and encouragement, at first, would suggest that electric energy can now be obtained in this favored district at very low rates, and, comparing the rates offered with the cost to any given consumer to produce power or light by steam-driven prime movers, certainly the rates do improve these conditions somewhat, as is shown by the steady growth of electrically-operated industries in Hamilton, Ont. Moreover, from $20 000 000 to $25 000 000 have already been invested by these four companies in generating stations and transmission systems which must find a return by the sale of the electrical output, but, for the last ten years, the people of Southwestern Ontario have steadily asserted that, to create a great industrial area in that part of the province, still cheaper power ought to be and could be obtained.

Boards of trade began to agitate in 1900, and in 1902 a committee, from various cities and towns, was formed to take concrete action, which, by pressure on the Provincial Government, induced the passage of an Act in 1903, enabling two or more municipalities to appoint a Commission and, after investigation, to carry out, if desired, the construction of the necessary works at Niagara or elsewhere for generating, transmitting, and distributing electric energy.

This municipal commission was appointed, made a very exhaustive investigation, and reported in 1906 to the seven municipalities then represented in the movement, but, owing to the limitations evident in the working of the Act, and to a larger movement having commenced, there has been no specific outcome from this report.

In 1905 the Provincial Government, becoming more fully alive to the importance of this question, appointed a Government Commission, which was instructed to study and report on the hydraulic possibilities in the province, the industrial demand then existing for electric energy, and the prime cost of generating and delivering the same to the various industrial centers and customers. The reports arising from the investigations were presented, in due course, to the Legislature during the session of 1905-06, and, although the facts elicited furnished a comprehensive idea of the industrial condition then existing, and the cost of satisfying the same, and also, by comparison,

set forth the rates offered by distributing companies, it has only been after a subsequent protracted struggle that definite results have been obtained and the evident will of the people carried out.

The vested interests immediately attacked the reports and the *bona fides* of those who drew them up, but the Provincial Government, recognizing the demand made upon it, passed a very comprehensive Act, appointing a permanent Government Commission. In this commission was vested the rights of the Crown, and it was authorized to act in response to the demands of the municipalities and in their behalf.

This Commission, after 2½ years' existence, has justified itself; and a group of sixteen municipalities has made a demand upon the Commission to secure electric power, to construct transmission lines from Niagara, to transform the electric energy thus transmitted at the various sub-stations, to administer the construction, and to operate the works for a period of 30 years, over which period a sinking fund will be created out of the rates to repay the investments. The amount of power to be distributed immediately is approximately 30 000 e.h.p., but it is confidently expected that this will increase very rapidly to probably 100 000 e.h.p. in 5 years.

The various municipalities have voted the money necessary for local distribution, and the Government has recently contracted for the construction of 300 miles of transmission lines, and this work is to be followed immediately by the construction of the necessary sub-stations and local distributions.

This popular movement has yet to be justified by capable and honest administration, deficiency in which is the weakness of many municipal movements in America, and it is to be hoped that the ægis of the Government Commission will render the inherent merits of the project effective.

The large area of country to be covered and the necessary anticipation of a still wider area as soon as the first installation has justified itself, have forced on the Government engineering staff, a consideration of higher voltage, and, according to present plans, the ultimate voltage of transmission now being provided for is 100 000. This advanced practice, however, has been adopted by reason of the necessary provision for further extensions of the Hydro-Electric Commission's system to points still more distant from the generating station

than London and St. Thomas (140 miles). The world will, doubtless, watch closely the results obtained in carrying out this huge municipal-provincial distribution of electric energy.

St. Lawrence River.—The St. Lawrence River, between Lake Ontario and Montreal, may be considered as having its navigation interests paramount, and, although attempts have recently been made to obtain permission from the Canadian Government to devote the whole river to power development, it is not likely that such permission will ever be granted.

In the meantime, along the ship-canal system, many small installations have been permitted, but they are chiefly local in character and are devoted to operating mills and factories. They amount roughly to 10 000 h.p., and, in addition to this, hydro-electric stations having a capacity of about 5 000 h.p. supply the various near-by towns with light and power, and light the ship canals during the navigation season.

The whole river, if available, would be a great source of power, as it is capable of developing, at low water, 1 500 000 h.p. of electric energy at the various rapids.

Montreal.—In the vicinity of Montreal (population 400 000), several interesting constructions have been made at various times by various companies, but finally all these interests have been merged into the Montreal Light, Heat and Power Company which controls the situation at present, although a rival company has recently been organized to develop power on the abandoned Beauharnois Canal of the St. Lawrence system and transmit to Montreal and distribute in opposition to the Montreal Light, Heat and Power Company. The latest construction by the latter company is at Soulanges, on the Soulanges Canal of the St. Lawrence River, 27½ miles west of Montreal, where 40 000 cu. ft. per sec. are available under a head of 54 ft. The equipment now being installed consists of three units, each of 3 750 kw., which will generate 3-phase, 60-cycle current at 4 000 volts. The transmission will be on one circuit at 44 000 to 40 000 volts, and the transformation will be made by banks of single-phase, water-cooled transformers having a rated capacity of 2 500 kw. per phase.

At present, this company owns two water-power stations and four steam stations, the latter having a capacity of 9 000 h.p. The com-

pany also purchases 16 000 h. p. delivered in Montreal by the Shawenegan Water and Power Company.

The station at Lachine, on the St. Lawrence River, is an example of daring but mistaken construction, carried out 12 years ago, when the transmission problem had not been as fully worked out as at

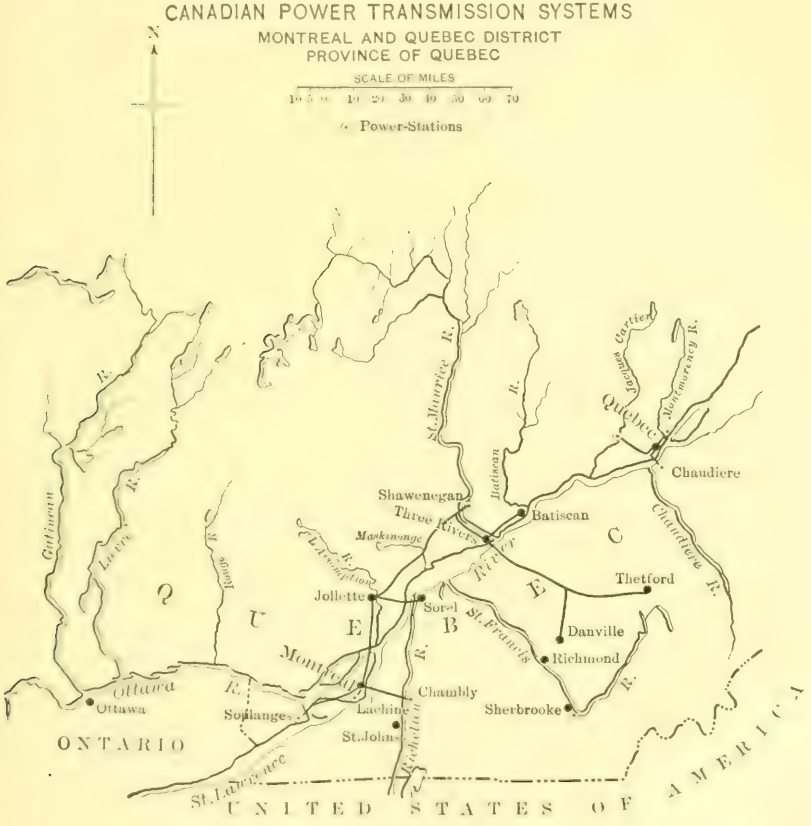

FIG. 5.

present. The works are really constructed as a part of the river, and therefore, in operation, they are subject to troubles from frazil in the head- and tail-races; but, after excessive expenditures, made for the purpose of creating quiet conditions above the power-house and protecting the tail-race from back-water, the station can be reasonably depended on to give continuous winter service.

The dependable hydraulic head is only 14 ft., and the eight generators, each having a rated capacity of 750 kw., are operated by groups of six turbines each, driving jack shafts.

The current generated is 3-phase, 60-cycle, at 4 400 volts, and is transmitted over eleven circuits at this voltage direct to Montreal (6½ miles), where it is stepped down to 2 400 volts for distribution. The station was designed for a capacity of 16 000 h.p., but it has never been equipped with machinery beyond 8 000 h.p.

At Chambly, on the Richelieu River, a branch of the St. Lawrence, 18 miles southeast of Montreal, the company owns a generating station which has also had a history interesting to engineers. The concrete dam, constructed to create a head of 34 ft., failed soon after completion, and the first year's operation was much disturbed by frazil, but a new dam has since replaced the faulty structure, and the station now operates normally.

The equipment consists of eight units, each of 1 800 kw., driven by four turbines per unit, the total generating capacity of the station being 20 000 e.h.p.; but the average load carried is 15 000 e.h.p. The current, which is 2 200-volt, 2-phase, 66-cycle, is stepped up to 25 000 volts, 3-phase, by 2 750 kw., single-phase, air-blast transformers, and is delivered over four circuits to Montreal, where it is transformed into the city distribution systems.

The complexity of distribution at Montreal may be appreciated when it is noted that the current is of three different frequencies, namely, at 30 cycles from Shawenegan, at 60 from Lachine, and at 66 from Chambly, to which must be added the ordinary conversions to D. C., and to various distributing voltages, A. C.

The total distribution of power in the city from the various stations of the company varies from, say, 30 000 h.p. in the summer to as much as 53 000 h.p. at peak load in the winter, and this amount will be supplemented soon by delivery from the Soulanges station now under construction.

St. Lawrence River Branches.—The St. Lawrence River Basin is traversed by many large tributaries which will be discussed briefly in respect to such developments as have been carried out, but it may be stated briefly that, although many small towns and mining and milling centers have hydraulic and electric stations of modest proportions satisfying present needs, there yet remains opportunity for developing

millions of horse-power on such rivers as the Ottawa and its tributaries, the St. Maurice, the Saugenay, the Trent, Spanish, Nepigon, etc., and projects innumerable of greater or less merit are being studied carefully by engineers and financiers with a view to meeting industrial needs as they arise, and creating new centers and industries to consume the available power.

The Kaministiqua Power Company has constructed a generating station at Kakabeka Falls, on the Kaministiqua River, and transmits power to the City of Fort William (population 15 000) at the head of Lake Superior, where grain elevators, flour mills, foundries, and municipal utilities consume some 6 000 or 7 000 h.p. at the present time.

The development has a hydraulic head of 175 ft., and carries the water through two 10-ft. concrete tubes a distance of 6 500 ft. to a compensating reservoir, and thence by steel pipes to the power-house. The equipment consists of two 4 000 kw., 3-phase, 60-cycle, 4 000-volt units, driven by German-built turbines, and the current thus generated is raised to 25 000 volts by two banks of single-phase, air-cooled transformers of 1 500 kw. per phase; it is transmitted 19 miles over two circuits of No. 00 copper on wooden poles to Fort William.

In 1904 the Canadian Copper Company (otherwise the International Nickel Company) constructed a generating station on the Spanish River at Turbine, from which current is delivered to its mines and smelters at Copper Cliff near Sudbury, which lies north of Georgian Bay.

The construction of a large head reservoir created good operating conditions, and increased the hydraulic head to 85 ft. Two 2 000-kw. units are installed and a third is now under construction. The current as generated is 3-phase, 25-cycle, at 2 400 volts, and is stepped up to 35 000 volts for transmission. It is carried 29 miles to Copper Cliff, by two 3-phase circuits of No. 1 copper mounted on a double-pole line.

The company has various sub-stations where the current is distributed for service in the mines and at the smelters, which it is understood turn out more than one-half of the world's consumption of nickel.

The Mond Nickel Company is also installing a 1 500-h.p. hydro-electric station for similar purposes on the Vermilion River, and the current is to be transmitted 11 miles to the Victoria mines and smelters, 25 miles southwest of Sudbury.

The Town of Orillia (population 6 000) has constructed a power-station on the Severn River, and transmits some 1 600 to 1 800 h.p., at 22 000 volts, a distance of 20 miles. It distributes the current for industries as well as for pumping and lighting; and, by offering low power rates, has established a stable industrial center.

The City of Ottawa (population 75 000) is in the midst of many developed and undeveloped water-powers, and the latter, in the event of the construction of the Georgian Bay Ship Canal, would become readily available at low investment costs. In addition to the Ottawa River itself, many of its larger branches, especially in the Province of Quebec, are very great latent sources of hydraulic power, and it may be stated conservatively that within a radius of 75 miles of Ottawa City there can be developed on the Ottawa River and its main tributaries 500 000 h.p. at low water, which amount would be increased materially by the construction of the ship canal.

Near or in the City of Ottawa there are ten hydraulic and hydro-electric developments operating under various heads, but chiefly on the Chaudière Falls at from 22 to 35 ft. These developments form the industrial center of the city, and operate very extensive pulp and paper mills on both sides of the river. There are also cement mills, carbide furnaces, and other industries, and power for street railways, lighting, and general city use is also produced. About 35 000 h.p. is now developed, and, as many conflicting interests were contending for their respective shares of the flow of the river, it became evident that some improvement was imperative, so that an agreement to construct a proper dam at Chaudière Falls has been entered into and the work is in progress.

This dam will increase the hydraulic head somewhat, and will drown out the rapids immediately adjoining, thereby lessening the very serious present difficulties in respect to operating in the winter season, when the water is full of frazil, and chiefly to conserve the last drop of water in low-water periods, thereby increasing the available power at such seasons. However, in the near future, more power will be required, and it is proposed that the city itself develop the splendid powers quite near it on the Gatineau River, where a head of more than 80 ft. can be secured at a point within 8 miles of the city, and where 50 000 h.p. can be developed at one location. Considering the vast timber and mineral resources tributary to Ottawa, and its position as

Capital of the Dominion, it may be considered certain that the availability of unlimited hydro-electric power assures the industrial future of the city.

Quebec.—The Ottawa is only one of many great rivers flowing into the St. Lawrence from the north between Montreal and the sea, and, as all these rivers rise in unbroken forests on the Laurentian highlands, and are all well supplied with splendid storage facilities, they are all valuable sources of power. However, the sparse population has been a deterrent, and the only large works north of the St. Lawrence, and devoted to the generation of electric energy for transmission purposes, is on the St. Maurice River at Shawenegan Falls, where the Shawenegan Water and Power Company has created a large industrial town and also generates a large amount of power for transmission to Montreal, Sorel, Three Rivers, Thetford, etc., and will doubtless soon enter the field at Quebec in opposition to the present companies.

The hydraulic conditions at this place are most favorable, as the head-works consist of a natural lake, which prevents trouble from frazil in the winter. The drainage area is some 18 000 sq. miles, and a flow of 10 000 cu. ft. per sec. is considered the minimum. This, at a hydraulic head of 130 ft., provides more than 100 000 e.h.p. at all seasons. The works have been carried on partly for the generation of electric energy and partly to operate local industries hydraulically. The products of these industries are pulp, paper, aluminum, carbide, etc.

The electric station has three 4 000-kw. and two 7 000-kw. units, generating 2-phase, 30-cycle current at 2 200 volts, and this is transformed to 50 000 volts by banks of 1 100-kw. and 2 200-kw. single-phase, water-cooled transformers having a total capacity of 32 000 kw.

The transmission circuits are of aluminum cable on wooden poles. The Montreal service (85 miles) has been in operation for the past 10 or 12 years, and has given good satisfaction.

The delivery in Montreal is by two circuits, and amounts to 16 000 h.p.; besides which, the company serves the asbestos and other mineral districts on the south side of the St. Lawrence at Danville, Thetford, etc. The accomplishments of this company within the last ten years have been considerable; indeed, its works form the basis of many large industries, and it can be confidently expected that the whole 100 000 h.p. available at this one water-power will be made use of in the near future.

While Shawenegan is an instance of the water-power being developed as a foundation for industries and to supply distant markets, the developments in the vicinity of Quebec City (population 90 000) have been made strictly to supply existing needs. The installation on the Chaudière River, south of the St. Lawrence, has a hydraulic head of 114 ft., and is laid out for 6 000 h.p., but the equipment, thus far, is for not more than one-half this amount. The current is 3-phase, 66-cycle, and is generated at 11 000 volts. It is transmitted to Levis and also to Quebec City by insulated cable laid on the bed of the river.

Two developments north of the St. Lawrence, on the Montmorency River and on the Jacques Cartier River, supply current for city lighting, street railway operation, cotton mills, etc.

The Montmorency works, having a capacity of 4 000 h.p., have five units operating under a head of 220 ft., and generate 2-phase current at 5 000 volts, which is transmitted direct over four circuits to the city sub-stations.

The Jacques Cartier works have 2 000 h.p. installed, and the current is transmitted 18 miles at 20 000 volts over two 3-phase circuits to the city limits, where the company has a steam auxiliary station of about 2 000 h.p. to carry peak loads.

Some Conclusions.

A careful consideration of the electrical data contained herein will indicate the trend of activity in Canada in hydro-electric investments and the reasons therefor. In general, it may be said that on the Atlantic Coast and over the western and central portions of the great plains very little benefit is to be derived from such constructions, but over all the Provinces of New Brunswick, Quebec, Ontario, Eastern Manitoba, Western Alberta, and British Columbia, the continued construction of additional hydro-electric stations and the distribution of electric current to the practical limits of transmission may be confidently predicted. The resources of Canada, in minerals, timber, and agriculture, are so varied and so immense that the growth of industry as a legitimate outcome of the development of these resources, and the corresponding increase in population, may be looked on as a foregone conclusion, and it is as certain that, in this industrial growth, low-priced power will form a large factor.

At present the hydro-electric developments serve to a greater or

less extent such industrial centers as Montreal, Quebec, Ottawa, Peterboro, Toronto, Hamilton, Winnipeg, and Vancouver, besides a great number of smaller ones; but only a beginning has been made. Probably the 500 000 h.p. now hydraulically developed would include only stations of 1 000 h.p. or more, and there are literally many millions of horse-power available for development within easy reach of the present population; but, aside from the increased industrial demand, there are specific demands of great importance. Already, as noted, several mining interests are served by hydro-electric power, and with the growth of mining investments the importance of such service will increase. Then again, pulp and paper (with the depletion of United States supplies) will become a much more important industry than at present, and consequently much hydraulic and electric power for 24-hour service will be thus consumed.

Particular attention, however, is directed toward the enormous opportunities for electro-chemical industry. At Niagara Falls, N. Y., a large group of such industries is now well established, and in Canada a modest beginning has been made at Shawenegan, Ottawa, and Niagara Falls, Ont.; but Canada has in its great rivers sources of power as cheap as any in the world, except that the labor item may increase slightly the investment per horse-power beyond that of a similar construction in Europe. The whole question rests on transportation and markets; the latter, artificially restricted by the United States tariff, must seek an outlet in Great Britain, and as it is well demonstrated that $10 per horse-power-year, continuous power, at the low-tension switch-boards, is a remunerative price for large installations, and will cover all fixed charges, depreciation, and attendance, there would appear to be a good field for investment.

The financial condition in Canada at present, as regards hydro-electric enterprises, is worth considering. Some 10 or 15 years ago many of the brighter minds in the country seized the idea that, by securing valuable long-term franchises, on the one hand, for electrical service, and on the other for water privileges on near-by rivers, a splendid opportunity existed for profitable exploiting.

The net result at present is that, in almost all the large towns and cities, these services are administered by companies whose charges for power and light are only regulated by two things: either the exigency of the occasion, or the cost of a similar service produced by the

customer from coal usually imported from the United States; and as a good quality, run-of-mine, bituminous coal costs from $3.50 to $4.00 per ton in Ontario and Quebec and $6.50 per ton in Manitoba, it is evident that such competition is harmless, and the electrical companies were and are able to do business at very profitable rates.

In recent years, however, the municipalities have become active, and several smaller towns and cities have constructed municipal hydro-electric stations. The movement is very active at present in the Province of Ontario, and to a less extent in other provinces.

The struggle is bound to be prolonged and bitter; indeed, much bad feeling and resort to disreputable tactics have already been exhibited. Municipal and provincial credit has been attacked in the London market, and all the influence which heavy vested interests can exert is and will be directed against further municipal action.

The ultimate result is evident; the public mind will become more and more inflamed by the obstruction of the electrical companies, and, undoubtedly, in the end, the municipalities will carry out the expressed wish of the majority; but in the meantime public credit is unsettled, the manufacturers are playing a waiting game, and no one is benefited because the expansion of the market, and consequent advantage to every one, including the electrical manufacturing companies, has not taken place.

It would appear advisable to consider the appointment of a Public Service Commission by the Dominion Government to regulate the rates which may be charged by companies already in operation, so as to secure that benefit to the public which the presence of abundant water-powers warrants, because it is not probable that, at the most, more than a majority of the industrial centers will be served municipally, and, judging by the results obtained by the Dominion Railway Commission, much satisfaction might be obtained and the public mind be set at rest if a rate-regulating commission for electric utilities, with wide powers of action, should be appointed.

Concurrently, however, with the distribution of electric power for utilization in cities, etc., it is to be anticipated that very large constructions will be undertaken in the near future to furnish power for the operation of railways over heavy grades, where coal is expensive or traffic dense, and that at the same time the natural resources of the country will develop to an increasing extent, and that

in mining work, pulp and paper manufacture, electric smelting, and general electro-chemical service will be found the most attractive fields for the enormous and not fully appreciated sources of cheap and widely-distributed power which is now wasting in so many districts of Canada.

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PAPERS AND DISCUSSIONS

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SAMPITTIC SURFACING.

Discussion.*

By Messrs. L. W. Page, Prevost Hubbard, Arthur H. Blanchard, Philip P. Sharples, H. W. Clark, C. E. Grunsky, and Henry B. Drowne.

Mr. Page.

L. W. Page, Esq.† (by letter).—While the writer is in hearty accord with the growing desire exhibited by road engineers to obtain standard specifications for bituminous cements to be used in road treatment, and while he realizes to the fullest extent the benefits which would be derived from such specifications could they be produced, there are certain points which are apt to be overlooked by the engineer and should receive serious consideration.

In the first place, local conditions, such as climate, location of the road, quantity and character of the traffic to which the road is subjected, as well as the character of the road material, have such an important bearing on the requisite characteristics of a bituminous binder that even when the same methods of construction and application are followed, it is doubtful if any one cement will prove equally satisfactory for all cases. In many instances experiment alone will determine what binder is best suited to meet given conditions, and the impracticability of following the same strict specifications for all cases, therefore, is apparent.

Broad specifications in regard to the elimination of certain objectionable properties which have invariably produced bad results,

*This discussion (of the paper by Walter Wilson Crosby, M. Am Soc. C. E., printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Director of Public Roads, Washington, D. C.

and favoring those characteristics which are essentially necessary to Mr. Page.
a good binding material, can be framed, of course, but strict limitations with respect to certain constituents, such as free carbon, fixed carbon, paraffin contents, etc., are, at the present time, based on matters of personal opinion rather than on established facts. This is particularly true of country roads, as considered apart from city pavements, and is largely due to two facts: (1) That, owing to necessarily less refined methods of construction and less severe traffic conditions, binders which would be of little service for city work will often prove satisfactory for country highway work; (2) that, at the present time, there are but few reliable data as to the characteristics of the binders which have been used in experimental work, and their effect on the results obtained.

From a careful consideration of this subject the writer is led to the conclusion that the framing of specifications for bituminous road binders is, and will be for many years to come, a matter of expert opinion for individual cases, and that, in view of the fact that the near future will see an immense amount of bituminous road work carried on, every progressive road engineer should make a study of bituminous road binders, and, in all reports of experimental work, should include the results of an examination of the binder which he has used. By so doing, any binder which has met his requirements in a satisfactory manner can be identified, and specifications can be made which will enable him to obtain a like material again and duplicate his results. His experience will also be of considerable value to others who are working along similar lines.

In regard to Mr. Crosby's proposed pitch compound, it might be said that, while there is little doubt that under certain conditions a material meeting his specifications would prove satisfactory, it is questionable whether such a guaranteed material could be generally obtained at anything like a reasonable cost. The time required to make an examination covering these specifications is another objectionable feature. It might be added that, while Section 287 states that "the pitch compound to be used shall be a hydrocarbon, refined from tar or petroleum, or made from natural asphaltic pitches by the use of a proper solvent or flux", according to the writer's experience practically all gas-house tar pitches are eliminated by Clause *a*, and even coke-oven and water-gas tar pitches by Clause *j*.

According to the writer's belief, any ductility test made as described in Clause *e* is of little or no service in determining the road-binding value of the original material, for the reason that, when subjected to temperatures sufficiently high to give a residue of the consistency stated, many oils are so altered that the residue thus artificially produced in no way represents the binding value of the original material.

Mr. Hubbard.

PREVOST HUBBARD, Esq.* (by letter).—Before discussing Mr. Crosby's specifications for a basic pitch compound, the writer wishes to state that his views relative to the practicability, or rather the impracticability, of framing strict specifications which will meet all conditions satisfactorily are in close agreement with those presented by Mr. Page.

Considered from a chemical standpoint alone, the wide differences in composition existing between the two classes of bituminous binders —tars and asphaltic oils—make it a difficult matter to deal with both in one set of specifications. This can readily be shown by taking up the various clauses separately and determining their value as applied to each class of binders. By so doing, a number of interesting facts pointing toward the most rational methods of examination for each class may also be developed. As a consideration of this subject in detail, however, would constitute a paper by itself, only those points which the writer believes to be most important will be included in the following brief discussion.

Clause a.—From the examination of a considerable variety of bituminous road binders in the Laboratory of the United States Office of Public Roads, it would seem that any specification limiting the percentage of free carbon to 10 is applicable to tars only. A total bitumen clause could be substituted for oils, and while the two determinations are made in much the same way, the amount of material insoluble in carbon bisulphide could be limited to 2% for most asphaltic oil products. In cases where a material such as Trinidad asphalt had been fluxed with an oil to produce the pitch compound, however, allowance would have to be made for a considerable amount of inorganic material which, of course, would be insoluble in carbon bisulphide. As applied to tars, a clause limiting the free carbon contents to 10% would bar out practically all gas-house coal tars, and, while the writer firmly believes in the superiority of low free carbon tars, it would seem hardly fair to eliminate those containing more than 10%, as this class constitutes a large proportion of the tars produced in the United States, and, in some instances, good results have attended their use in road construction.

Clause b.—A determination of fixed carbon in a bituminous road binder is of value mainly for the purpose of determining the mechanical stability of the bitumens present. As this property, within certain limits, is believed to be almost directly proportional to the percentage of fixed carbon, it would seem that if any limitation is made it should be in the other direction, that is, the lowest limit should be stated rather than the highest. When applied to the bitumen itself, it is extremely doubtful if any ordinary pitch compound would show fixed carbon as high as 20%, and, if it showed as high as 15%, it could

* Assistant Chemist, Office of Public Roads, U. S. Dept. of Agriculture.

hardly be expected to meet the requirements of Clause *g*. In addition, it might be said that while an oil or asphaltic product with fixed carbon running as high as 7% might be satisfactory for construction work, the same would not necessarily be true of a tar pitch, even if the free carbon content were not considered. A fixed-carbon specification, therefore, would seem to be of value mainly as applied to oil and asphaltic products, in the manner indicated. Mr. Hubbard.

Clause c.—As there is considerable difference of opinion among chemists, at the present time, as to the value of a paraffin determination applied to road oils, the writer will not consider it in detail. It might be stated, however, that, without question, such a determination is unnecessary in the examination of tars. The writer also believes that, as a rule, it is unnecessary in the examination of oils, as the characteristics of the base obtained from a volatilization test will show whether paraffin hydrocarbons are present in sufficient quantity to injure the material for road use.

Clause d.—The main objection to this clause is that a 24-hour continuous volatilization test at a temperature of 105° cent., is impracticable for most laboratories.

Clause e.—While no criticism of a ductility test, if it can be made on the original material, is here intended, it would seem to be of little or no value as applied to residues produced at high temperatures, for the reason that in producing such residues many bituminous materials are chemically altered in such a way that a ductility test made in the manner described cannot be depended on to represent the characteristics of the original material. This can often be shown by a comparison of the relative fixed carbon, naphtha insoluble, and carbon tetrachloride insoluble determinations made upon the original materials and the residue.

Clause f.—This clause will not be discussed, owing to its direct relation to Clause *d*, which has already been considered.

Clauses g, h, and i.—A determination of viscosity and melting point is undoubtedly of considerable value when it is desired to apply a material in a given manner and under given conditions; but the writer believes that a determination of the consistency of a residue resulting from a volatilization test made at comparatively low temperatures, say, 163° or 205° cent., is of more service than that of the percentage of residue of given consistency which can be produced at high temperatures. The reasons for this are the same as those mentioned under Clause *e*.

Clause j.—While this specification might be applied to oil products, it would bar out practically all tar preparations which, as a rule, contain considerably more than 20% of oils which can be removed by distilling between 170° and 270° cent.

Clause k.—The only criticisms applicable to this clause have been made evident in the discussion of Clause *a*.

Mr. Hubbard. *Clauses l and m.*—These clauses would seem to be applicable and necessary to any set of specifications, but it should be remembered that under ordinary conditions where a carload of material is to be held up until an examination can be made, a very limited time can be allowed for such an examination. From this point of view, alone, it is evident that an examination, according to Mr. Crosby's proposed specifications, requires entirely too much time to be practicable.

Conclusions.—In general it would seem inadvisable to attempt to frame specifications for bituminous road binders, except for individual cases. Somewhat different methods of examination are required for asphaltic oil products and tar products. Owing to the fact that no standard methods of examination have as yet been established for these materials, reports from different chemists upon the same material are apt to differ greatly. The establishment of standard methods of examination, therefore, should receive first consideration; otherwise, exact descriptions of the methods which should be followed will have to be included in each set of specifications. Owing to lack of detailed data as to the exact effect of composition upon the results obtained in practice, an examination of bituminous road binders is mainly of value as a means of identification and for the purpose of determining which of a number of materials is best adapted to meet given conditions. Strict specifications, therefore, cannot be made satisfactorily until such data are available. In future experimental work with bituminous road binders, therefore, it is recommended that the results of an examination of the binder be included in any report which may be made.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Highway engineers owe Mr. Crosby a debt of gratitude for presenting for criticism, tentative specifications covering pitch compounds, and thus precipitating a discussion of bituminous materials in its entirety.

The present status of specifications covering bituminous materials is unsatisfactory from the standpoint of both the producer and the purchaser, and is due primarily to a lack of knowledge of the methods of specifying the requisite properties, and to the insufficiency of published data relative to the results attained by using various bituminous materials in divers ways.

Without doubt, every highway engineer dealing with the construction and maintenance of roads subjected to excessive motor-car traffic will welcome enthusiastically the adoption of standard specifications which will permit of calling for bids for a bituminous binder, with the expectation that the material submitted will be adapted to the exigency. The attainment of this desideratum will take from two to five years of co-operative work on the part of chemists and of the highway engineers who have charge of the construction and maintenance of bituminous surfaces.

Mr. Blanchard.

One method of procedure which would result in the acquisition of the desired data in the near future is as follows: First, keep full notes during construction, covering the points mentioned below; second, make a complete chemical and physical analysis of the bituminous material in accordance with standard methods; and third, keep a complete record of the condition of the road under varying climatic conditions. By compilation and a comparison of the data acquired it will be possible, finally, to determine the essential requisites for bituminous material to be used in any particular method of construction, and the requirements which must be included in specifications in order to secure materials which possess these properties.

The essential data to be recorded covering the construction of bituminous surfaces are as follows:

ROAD PRESERVATION.

Data Relative to Construction.

State
County.
Town.
Name of highway.
Class of highway.*
Nature of traffic.*
Traffic census.†
Distance to nearest railroad station.
Distance to source of materials.
Length.
Width.
Area, in square yards.
Nature of subgrade.
Maximum depth and length of fill.
Maximum, mean, and minimum grades.
Kind of stone, gravel or sand used.
Sizes or grades of broken stone or gravel.
Contractor.
Dates when construction was commenced and completed, or months during which construction was carried on.
Temperatures at 8 A. M. and 2 P. M. on each day during construction.
Climatic conditions on working days.
Location of car tracks, if any, relative to bituminous surface.
Road exposed or protected.
Number of hours in legal working day.
Average rate of progress per day, in square yards.
Specifications covering construction of bituminous surface.
Details of construction.
Data relative to contract covering the construction of the road complete.
Bituminous material furnished by contractor, or by party of the first part.

* See *Transactions*, Am. Soc. C. E., Vol. LXI, p. 446.
† See *Engineering News*, Vol. 61, p. 466.

Mr. Blanchard.

Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard complete.
Estimated cost per square yard over and above cost of similar construction minus bituminous material.
Data relative to contract covering the construction of the bituminous surface as a secondary contract.
Lump sum contract, or price per square yard.
Bituminous material furnished by contractor, or by party of the first part.
Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard of bituminous surface complete.
Day labor plus percentage contract.
Bituminous material furnished by contractor, or by party of the first part.
Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard of bituminous surface, complete.
Rebate on watering cart, labor, screenings, etc.
Percentage allowed on extra work.
Unit prices (hour basis).
Common laborer.
Foreman.
Man capable of handling bituminous material.
Double team.
Single team.
Rental of mixing plant, kettles, and accessories, distributor, or other special apparatus.
Roller.
Watering cart.
Scarifier.

Every effort should be made within the next two months to standardize methods of determining chemical and physical properties, in order to obtain the maximum amount of information from the construction work of 1909.

In the laboratory of the State Board of Public Roads of Rhode Island, tests are made to determine the following properties in accordance with methods adopted by Mr. Crosby and the writer:

Tars.—
Water-soluble materials.
Specific gravity.
Free carbon.
Fixed carbon.
Loss by evaporation for 21 hours at 105° cent.
Penetration of residue.
Melting point of residue.
Loss by distillation.
Viscosity.

Mr. Blanchard.

Asphalts and Asphaltic Oils.—
Melting point of solid asphalts.
Penetration of solid asphalts.
Water-soluble materials.
Specific gravity.
Free carbon.
Fixed carbon.
Paraffin.
Loss by evaporation for 7 hours at 170° cent.
Loss by evaporation for 21 hours at 105° cent.
    Penetration of residue.
    Melting point of residue.
Loss by evaporation for 21 hours at 225° cent.
    Penetration of residue.
    Melting point of residue.
Solubility in 88° Baumé naphtha.
Viscosity.

Other tests which are made in the Rhode Island laboratory are for the determination of the cohesive strength of the material under gradually applied tension, and the cohesive strength under impact.

The machines used in the Rhode Island laboratory for testing the cohesive strength of bituminous materials were designed by Henry B. Drowne, Assoc. M. Am. Soc. C. E., Assistant Engineer, State Board of Public Roads, and he will describe these machines and the methods of conducting the tests.

The object of the cohesion tests is to acquire information relative to the comparative value of various bituminous materials from the standpoint of their ability to resist: first, the action of motor-car traffic on the surface layer; second, the impact of horses' hoofs; and finally, to devise specifications covering these essential properties.

In writing specifications for bituminous materials to be used in highway construction, two facts must be kept constantly in mind, if such specifications are to prove satisfactory: first, the relationship between a given requirement and the value of that property of the material when considered from the standpoint of a road binder; and second, the advisability of limiting the requirements of the specifications to those for which the tests can be made in a reasonable period.

Mr. Sharples.

Philip P. Sharples, Esq.*—In discussing the subject of tests for bituminous materials, a sharp line should be drawn between methods of analysis suitable to be inserted in a specification and those which are of scientific or academic interest. Both the consumer and the manufacturer desire that a product with certain characteristics should be defined. The tests which shall confine the product within the limits should be as simple as possible and easily executed in any works laboratory without special apparatus.

* Chemist, Barrett Manufacturing Company, Boston, Mass.

Mr. Sharples. To confine this discussion to a criticism of the tests for tar materials, the specification, as set forth by Mr. Crosby in its amended form, requires far too many tests, and some of them are much more complicated than necessary. Mr. Smith has already outlined the objections to the tests, and Mr. Hubbard's discussion is along the same lines.

Mr. Crosby's specifications call for fifteen different tests. A specification calling for not more than six or seven tests should define the product with sufficient clearness. As a model, in the direction of simplicity, the specification of the Massachusetts Highway Commission for the recently advertised Milford work is to be commended for its form, although the figures are subject to modification. Mr. Clark discusses the Massachusetts specification, and a few comments are offered.

In the case of Mr. Crosby's tests, as Mr. Smith has pointed out, the free carbon content required is too low. A definite minimum and maximum should be set. The test for evaporation loss should be modified so that it may be compassed in one working day. Equally good results may be obtained, it is believed, by shortening the time and raising the temperature slightly.

The viscosity test should be made on a standard instrument, and preferably should be modified so that it will indicate the viscosity at ordinary temperatures.

To conclude, the speaker believes that the following tests are sufficient to define a coal-tar pitch compound: Specific gravity; free carbon; distillation test, including tests of oils and pitches obtained. To these may be added a viscosity or penetrometer test.

Mr. Clark. H. W. Clark, Esq.*—The speaker has been very much interested in Mr. Crosby's paper, and is glad to take part in the discussion. As has been stated by the author, the paper differs, in many respects, from the form in which the speaker first saw it, that is, as now presented, the limiting figures in the specifications have been omitted.

During the past nine months the Highway Commission of Massachusetts has had under way many laboratory and road experiments on this subject, and a large number of samples have been examined chemically and submitted to physical tests. Up to the present time, about 250 analyses of tars, refined tars, asphalt oils, oil asphalts, trade compounds, etc., have been made, and, as the result of all this work, certain specifications have been drawn up in regard to tests which these road materials should pass to show certain qualities which they must possess in order to make them of value in road work. In this experimental work and in drawing up the specifications the Commission has had in mind no theoretical material which, perhaps, might be of greater value as a binder than any now on the market, such a

* Chemist, Massachusetts State Board of Health.

material as probably would have to be found if all Mr. Crosby's specifications, as first presented, were held to; but, rather, an endeavor has been made to find out, from experimental roads and analyses of materials, which of these binding materials in each class has thus far, apparently, given the best results, and the specifications have been framed so that only the best of these materials, judging from chemical analyses and practical results, will pass them. It has been thought necessary and most practical to have separate specifications for refined tars, asphalt oils, oil asphalts, and natural asphalts, and in compiling these specifications the aim has been to make them as simple as possible, that is, not to make the labor of analytical chemical work and physical tests in various laboratories unduly arduous, but to include such tests as will surely, from what is known at present at least, cause the use of desirable binders. Mr. Clark.

The methods of analysis used by the Massachusetts Highway Commission differ to some extent from those outlined by Mr. Crosby. They are simpler, and it is intended to make them public soon, in order that those submitting binding materials can know just how the laboratory work of the Commission is done. It is not necessary to enter into a discussion of these methods, however, although attention may be called to one or two points. Considerable has been said in regard to physical tests, especially in the way of suggesting that tests similar to those to which cement is subjected may be eventually developed. It has been believed from the beginning that some work of this sort was necessary, and, for the past nine months, briquettes have been made of certain proportions of sand and the bituminous materials undergoing examination, and these briquettes have been submitted to certain tests. This was started coincidently with the chemical work, as it was believed that materials proposed as binders should show some binding property, and this property can certainly be shown more quickly by physical than by chemical methods. Of course, the chemical analyses are of more importance in many respects than any physical test. The specifications which the Highway Commission has just issued, include a briquette test. In studying these briquettes, however, it is fairly evident that it does not necessarily follow that those binders which soonest produce hard and strong briquettes are the most valuable. In Mr. Crosby's paper, as first presented to the speaker, there was one specification covering the loss which the material should undergo when 20 grammes were heated in a flat-bottomed dish, etc., for a certain number of hours. In its work the Massachusetts Highway Commission has found that the various classes of materials must be differentiated in this respect, for the refined tars and asphalt oils act very differently one from another in this question of loss of weight, and that, on the other hand, the oil asphalts may, and often do, gain in weight when kept at a high temperature for a prolonged period.

Mr. Clark. The type of viscosimeter used is believed to be practical and simple, but no hard-and-fast rule can be given in regard to the absolute time which a good binder must take in passing through a capillary tube of a certain diameter, equally good binders, as far as has been discovered, showing great variation in this respect, and in the specifications for each class of material quite wide limits of viscosity have been allowed.

Mr. Crosby appears to have made no statement in regard to specific gravity, but it has been found that the best materials of the different classes should have a certain specific gravity, if they are to be of value. For instance, the specifications of the Massachusetts Highway Commission state that the asphalt oils must have a specific gravity of 0.97. Oils with a specific gravity much less than this are light oils, with very little binding power and very little bituminous residue. They may be of some value in treating old roads, perhaps, but they are probably of little value in construction. To illustrate the simplicity of the specifications of the Massachusetts Highway Commission, those for refined tar and asphalt oils are included here. As stated previously, however, it is not to be supposed that the specifications cannot be improved, and that they will not be improved by the Commission eventually, but it is safe to say that any refined tar answering to this set of specifications will be included, as nearly as can be determined, in that class which gives the best results in road work.

Suggested Specifications for Refined Tar.

The tar shall be uniform in color, character, appearance, and viscosity, and shall have the following qualities:

1.—It shall contain not more than 0.5% of mineral matter or dirt.

2.—It shall have a specific gravity between 1.18 and 1.25.

3.—It shall not contain more than 14% by weight of free carbon.

4.—It shall contain no body which distils at a temperature lower than 225° cent.; not more than 10% by weight shall distil below 270° cent., and it shall contain at least 65% by weight of pitch or bituminous material remaining after all bodies up to 360° cent. have been distilled.

5.—When 20 grammes are heated in a flat-bottomed dish, 3 in. in diameter, for twenty-one hours, in an oven kept at a temperature of 100° cent., the loss shall be not more than 10% by weight.

6.—It shall be of such viscosity that 60 cu. cm., measured at room temperature (78° Fahr. or 26° cent.), shall, when at 100° cent., be not less than 85 sec. and not more than 240 sec. in passing through a viscosimeter orifice $\frac{5}{64}$ in. in diameter, when acting under a head of $4\frac{1}{4}$ in.

7.—When $12\frac{1}{2}$% by weight of the material is mixed with $87\frac{1}{2}$%

Mr. Clark.

by weight of sand, of such grade that all of it will pass through a sieve having 10 meshes to the linear inch, and that practically none of it will pass through one with 190 meshes to the linear inch, and briquettes, 3 in. square and $\frac{1}{2}$ in. thick, are made, such briquettes will harden to such an extent in seven days, at ordinary room temperature, that when they are laid flat and held by their edges by two parallel knife-edge bars, they shall not bend when a weight is suspended from a third knife-edge or parallel bar placed across their center, until this weight reaches 200 grammes, and they shall not break at less than 250 grammes, and the weight which causes bending shall not be greater than 80% of the weight which causes breaking.

SUGGESTED SPECIFICATIONS FOR ASPHALT OILS.

The oil submitted shall be of uniform color, appearance, general character, and viscosity. It shall contain no bodies not naturally present in an asphaltic oil, and shall fulfill the following requirements:

1.—It shall not contain more than 0.5% of dirt or adventitious mineral matter.

2.—It shall have a specific gravity of at least 0.97.

3.—It shall not contain more than 1% of matter insoluble in carbon bisulphide, and shall not contain more than 10.0% of matter insoluble in petroleum ether.

4.—It shall contain no body which distils at a temperature lower than 250° cent., and shall not lose more than 55% by weight by distilling to 360° cent.

5.—It shall be of such viscosity that 60 cu. cm., measured at room temperature (78° Fahr. or 26° cent.), shall, when at 100° cent., be not less than 5 min. nor more than 10 min. in passing through a viscosimeter orifice $\frac{5}{64}$ in. in diameter, when acting under a head of $4\frac{1}{2}$ in.

6.—When 20 grammes are heated in a flat-bottomed dish, 3 in. in diameter, for twenty-one hours, in an oven kept at a temperature of 100° cent., it shall not lose more than 5% by weight.

7.—When $12\frac{1}{2}$% by weight of material is mixed with $87\frac{1}{2}$% by weight of sand, and briquettes 3 in. square and $\frac{1}{2}$ in. thick, are made, these briquettes shall keep their shape and show some bond.

Mr. Blanchard's remarks in regard to mixtures of tars or refined tars and asphalts are very interesting. The fact that he builds the beautiful roads, which everybody seems to testify to, with this mixture, is simply another illustration that theory and practice do not always

Mr. Clark. agree. Theoretically, Mr. Blanchard's roads should not be so perfect with this mixture, as the two bodies, when mixed, do not combine, as can be seen easily by examination under a microscope, and if the proportion is not right, a spongy, crumbling mass results, and briquettes made of sand and this mixture have no cohesion whatever. The fact stated by Mr. Blanchard that the asphalt comes to the surface and forms a surface coating, simply shows that the two bodies are not combined, but simply mixed, and the asphalt separates itself from the tar. Judging from the statements made, it seems to be one of the ways, and, perhaps the best way, of getting a surface coating of asphalt.

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—Prior to 1900 practically no attempt had been made to prescribe the quality of bituminous materials to be used for paving purposes in San Francisco, Cal. The first bituminous rock pavements (rock asphalt) were laid about 1887. The specifications were silent as to the physical or chemical properties of the asphaltic material, the most that was prescribed being a minimum percentage of bituminous material. Fortunately, the materials which were then available were admirably adapted for paving purposes; the sand impregnated with the asphalt was of good quality, well graded, and, as has been demonstrated under the test of actual use, the asphalt was of a quality in every way suitable for paving purposes. It is not surprising, therefore, that the paving contractors succeeded, by a judicious mixing at the quarries of materials of various degrees of hardness, in furnishing rock asphalt serviceable alike under the hot summer sun in the interior cities of California and under the cool breezes of the Bay City.

Notwithstanding several failures of asphaltic wearing surfaces at Sacramento, there was put down, in 1888, a pavement in the rear of the State Capitol, under the advice and supervision of the writer, which, after 20 years, is still in fine condition, and on which there have been no repairs. The traffic, however, is not heavy.

The generally satisfactory quality of the bituminous materials used in California in street and road work is well recognized. The following, condensed from reports which the writer made in 1903, referring to the bituminous rock pavements of San Francisco, bears upon this fact. Some of these pavements were already more than 15 years old. The aggregate length of streets paved with bituminous rock was then 107 miles, and this mileage included many down-town streets having heavy traffic. The experience, since the first of these pavements was put down in San Francisco, indicates that about $200 per mile per year, with an additional allowance for re-surfacing annually an area equal to one-tenth of the roadway, the wearing surface of which is more than 10 years old, would keep the streets in good repair. This was not the amount actually expended for repairs. Many streets were in a neglected condition. The actual appropriation for repairs had

fallen about $80 000 behind the amount that should have been expended, and the current appropriations for the purpose were only about one-half of what was required to keep the streets in good condition. Mr. Grunsky.

At that time it was estimated that pavements of this material, when more than 10 years old, would need minor repairs at a cost of about 1⅛ cents per sq. yd. per annum, and that, in addition thereto, the wearing-surface renewal cost would be about 3⅝ cents.

Several years prior to the date to which the above remarks refer, the City of San Francisco, under the writer's supervision, had established a testing laboratory. Plans were made for a careful examination of the asphaltic materials offered for use, as well as of asphalts extracted from pavements which had been in service for some time. It was hoped that such an examination would enable the chemist to detect those properties of bituminous material which were essential in making it valuable as a binder in the wearing surface of a street or road. No information is at hand to show whether or not this work has been carried forward successfully.

The abundance of bitumens and of oils with an asphalt base in California, which are admittedly serviceable in road-building, has rendered the use of coal-tar products out of the question in that State. Not even the by-products of oil distillation were willingly accepted for use on city streets. On this subject, in 1902, as City Engineer of San Francisco, the writer called attention, as he had done in previous official reports, to the desirability of permitting the general use of oil asphalt in paving work. The view was taken that California is a great producer of oil, and, therefore, as far as practicable, the home uses of the products of the oil industry should be demonstrated and extended. The serviceability of oil asphalt as a substitute for the natural solid asphalt was not determinable short of a practical test.

"Its use on a few of the city streets, under various conditions of traffic and on different kinds of foundation will, it is hoped, go far toward determining the question of serviceability. It may be stated in this connection that the California oils nearly all have an asphalt and not a paraffin base, and that, therefore, the possibility of producing an asphalt, in the course of the oil-refining process, that will have properties practically identical with the natural solid asphalts, must be recognized."

It was pointed out that the market was stocked with the products of many refineries, that the origin of the products was not always traceable, and that even when the refinery was known, the source of the oils and the character of their mixtures might remain in doubt. It was also stated that at least five years should elapse before final judgment can be passed on an asphaltic paving material, and that, in some instances, even 10 years might be required to reach a verdict.

In the meantime the use of oils, and particularly of the heavy oils of the California type, as a binder in sampittic wearing surfaces has

Mr. Grunsky. been rapidly extended, and the serviceability of California oils for such use has been demonstrated.

It is not to be hoped that there can be devised any laboratory process which will reveal fully the value of any untried bituminous material as a binder. This value will depend not only on a satisfactory condition of the road surface at the time of its completion, but also on its behavior under traffic, and this, in turn, will depend, among other things, on the nature of the rock particles which are cemented together, and mainly on the life of the bituminous binder. This useful life cannot be predicted for untried material until possibly after a sufficient number of successes and failures in actual practice have been recorded, together with adequate information relating to the properties of the asphaltic materials disclosed in the laboratory, and of all conditions under which such materials entered into the work. It may then be possible to write specifications so that they will exclude all undesirable materials while admitting all those which are known to be good. It appears to be hopeless at this time to devise laboratory experiments or tests of the quality of substances of this class which, with the information thus gained, apart from actual use, will determine their paving value. The admission of untried materials, therefore, will always be with more or less risk of failure. Moreover, it follows from this that laboratory tests prescribed by specifications should be simple and require but little time, and that no attempt need be made to imitate in the laboratory the conditions under which the materials are to come into use. The old method of determining the petrolene and asphaltene contents of the bituminous compound, and then determining the viscosity, ductility, adhesiveness, hardness, and behavior in water, and under temperature changes, of a distillation residuum is just as likely to lead to a form of reliable specifications for materials of this class as a more complicated examination.

While the materials which are offered are being tried out on roadways, however, full records of their chemical and physical properties should be preserved, and the methods of ascertaining these properties should be standardized, so that from year to year the results may be compared. This is a matter which should not be confounded with the prescribing of simple yet adequate laboratory tests to appear or to be referred to in specifications for road work. The thorough discussion of this paper will be welcomed by all highway engineers, and it is to be hoped that the author's efforts to secure a standardization of laboratory processes and of specifications will bring about the desired result.

California is in the movement for good roads. The State is soon to vote on a proposition to issue bonds to the amount of $18 000 000 for new roads. One county is now expending $3 000 000 on its roads, and, in another county, a bond issue of $1 800 000 has been voted within the last few months for a similar purpose.

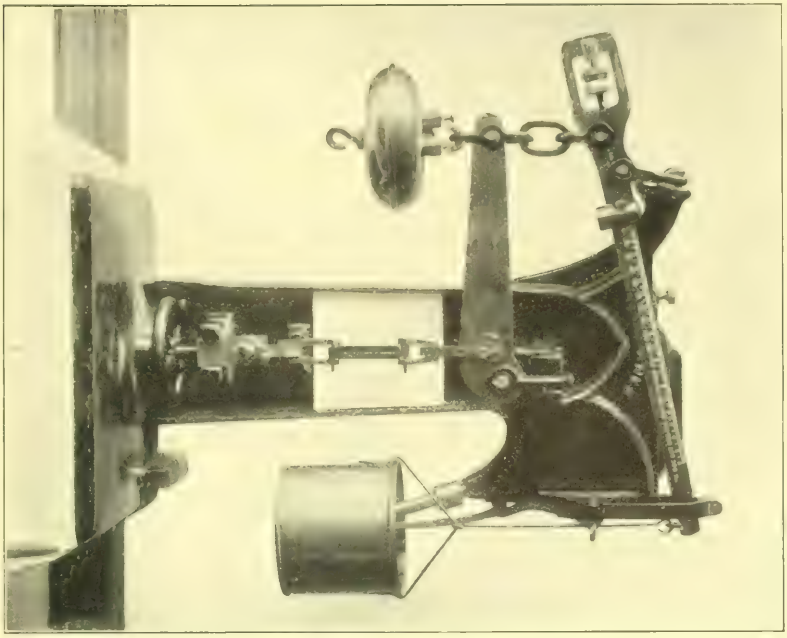

FIG. 1.—CEMENT TESTING MACHINE, MODIFIED FOR MAKING COHESION TESTS.

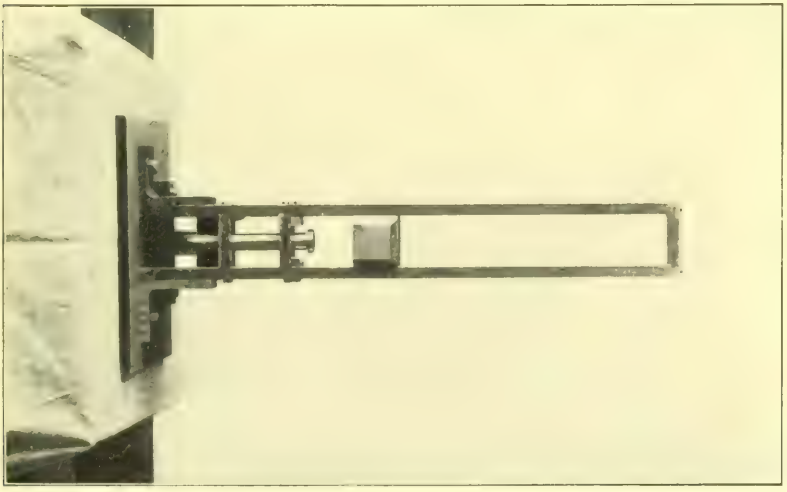

FIG. 2.—IMPACT MACHINE.

HENRY B. DROWNE, ASSOC. M. AM. SOC. C. E. (by letter).—It seems to the writer that at the present time no very rigid set of specifications as to the essential qualities of some of the bituminous materials now on the market can be written. Chemical and physical tests will show that a bituminous material possesses certain characteristics, but whether or not these characteristics are detrimental or beneficial can only be ascertained by observation of the action of the material in the road for a period of some years. It is important now, however, to develop tests by which the materials can be analyzed, and to have enough tests, and of such a character as to account for all phases of the action of the material, from the standpoint of both construction and service. In order that work in different parts of the country can be compared, it is of the utmost importance that uniform methods of testing bituminous materials should be adopted, and the writer believes that Mr. Crosby's paper and its discussion will aid in bringing about this result. Mr. Drowne.

In the Rhode Island laboratory, the engineers have been trying to develop two physical tests of bituminous materials to show the cohesiveness of the different materials under uniform tension and sudden shock. For convenience, the test with the steady pull is called the cohesion test, and that with a sudden shock, the impact test. A series of experiments was made on 1-in. cubes of stone cemented together with different bituminous products. The methods used in making the tests were rather crude, but the results showed that with different appliances a satisfactory test might be expected. After some study, it was decided to use metal blocks for the impact test and metal plates for the cohesion test. Metal was chosen because the same surfaces could more easily be produced, the metal is easily cleaned, and is durable; whereas, with stone, the kind of stone, its structure, and porosity would all be factors which might affect the results of the tests.

A Fairbanks cement testing machine, No. 3 194, is used to provide the uniform tension required in making the cohesion test. This machine was selected because many of the State highway and other laboratories are equipped with one. The modifications necessary to adapt this machine to the cohesion test are simple and readily duplicated, as is shown by Fig. 1, Plate XX. The two clamps which hold the cement briquettes are removed, together with the cast-iron "U's" by which they are attached to the knife-edges and pin. These fittings are replaced by bronze castings of a different and lighter pattern.* As these castings are lighter than the ones removed, sufficient weight must be added to the large ball to bring the scale beam to a true balance. The test plates are bronze castings of such a size that a

* Some of the illustrations for this discussion were not received in time to be reproduced in this number of *Proceedings*. They will be shown when the paper is printed in *Transactions*.

Mr. Drowne. surface of $2\frac{1}{2}$ sq. in. can be obtained. A shoulder is provided on the plates, so that when they are hung in the testing machine there will be no eccentricity of pull. The plates are cemented together, and are then connected to the upper and lower castings and tested.

The impact machine is practically a miniature pile-driver, as shown by Fig. 2, Plate XX. The essential parts are the leads, the 2-lb. weight, the plunger and guides, the clamp to hold the test blocks, and the stops on the leads to catch the weight when the specimen fails. The test blocks are 1-in. bronze cubes. The blocks are cemented together and placed in the clamp so that one of the blocks comes under the plunger. The weight is then raised and allowed to drop on the plunger; the number of blows are noted, and also the height of fall of the weight that causes failure.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE DESIGN OF ELEVATED TANKS AND STAND-PIPES.

Discussion.*

By Messrs. Charles B. Burdick, O. E. Selby, and Charles C. Hopkins.

Mr. Burdick. Charles B. Burdick, Assoc. M. Am. Soc. C. E. (by letter).—The author is to be commended for presenting in concise form the requirements for the design of elevated tanks and stand-pipes. The value of the paper, however, would be greatly increased by taking up the items separately and giving the reasons which led to the conclusions.

In regard to Section 7, on Unit Strains, apparently no especial consideration is given to the action of ice, which is generally regarded as producing an important stress. In a measure, this is allowed for in the moderate permissible tensile stresses in plates, and also in the requirement for minimum thickness of plates, which gives increased strength at the top of the tank where the ice action is probably at a maximum.

It seems to the writer, however, that the matter of climate should be given consideration in fixing allowable unit stresses. Good design will not permit the same strains in such extremes of climate as Florida and Minnesota. It is considered good practice in the vicinity of Chicago and northward to limit the plate unit stress to 10 000 lb. per sq. in. of net section for tanks in which the plate is directly exposed to the weather. It is believed, however, that proper design permits greater unit stresses where the steel is enclosed or encased by a protective covering which eliminates or reduces to a minimum the formation of ice.

* This discussion (of the paper by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for March, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Selby. O. E. SELBY, JUN. AM. SOC. C. E. (by letter).—This paper presents a desirable addition to the list of standard engineering specifications. Heretofore, specifications for structures included within the scope of the paper have been, to a great extent, the output of designers employed by manufacturers, and have shown the effects of the latter's bias.

If this discussion seems to be largely in the line of pointing out shortcomings in the author's specifications, it is not done in any spirit of criticism, but with a desire to see the specifications properly amplified and completed before they pass into a form which may be adopted as a standard. It is the writer's view that a technical specification should be something more than a mere check on the contractor; it should be a manual of design, complete enough to be a guide for the designer, whether he be employed by the purchaser or manufacturer, and should be so explicit and unambiguous as to permit of little discretion on the part of the designer, to the end that two independent designers working from it will produce substantially the same results.

The specifications seem to start with the assumption that the general dimensions of the structure are fixed. This should not be. For any given capacity and pressure there are certain economical ratios between height and diameter for elevated tanks, and there are other ratios for stand-pipes. Something relating to these rules of proportion should be included.

The specifications cover only stand-pipes and elevated tanks with hemispherical or flat bottoms supported on steel towers. One important form of elevated tank is ignored, that is, the tank with a semi-elliptical bottom and a small stand-pipe extending from the bottom to the ground, which serves both as a partial support for the tank and as a frost-proof inlet. This form is used largely for railroad water-station tanks, and the features which are peculiar to it should be included in this manual of design.

It is convenient to assume the weight of water at 62.5 lb. per cu. ft., because this makes convenient figures for calculation, namely, $8\frac{1}{3}$ lb. per gal. and $7\frac{1}{2}$ gal. per cu. ft. The use of two-thirds of the exposed area for computing wind pressure on cylindrical tanks is better practice than the use of one-half, as is done in many specifications. This ratio should be extended also to conical surfaces.

Table 1 allows a tension of only 12 000 lb. per sq. in. in tank plates, while a tension of 16 000 lb. per sq. in. is allowed in other parts of the structure. This is in accordance with common practice, and gives good results, although there is no apparent justification for the reduced unit stress. The tank plates are just as strong as structural shapes, and the loads coming upon them are just as capable of exact calculation. Of course, the ordinary reason for using a reduced unit stress is to provide extra section to cover corrosion and other forms

of deterioration. It would seem more rational to provide for this by an arbitrary addition to the thickness computed from the static pressure and net section. In fact, the writer has found it desirable and not any great hardship to specify an additional thickness of $\frac{1}{16}$ in. for tanks designed for a unit stress of 12 000 lb. Mr. Selby.

The allowable shear on rivets is given as 12 000 lb., with no stipulation that there shall be any reduction of this unit stress for rivets in tank plates. If the tank plates are designed for 12 000 lb. per sq. in. in tension, the rivets certainly ought to be designed for proportionately low unit stresses in shear and bearing. The allowed shear on field rivets and bolts is lower than that usually permitted. It is customary to prescribe that the number of field rivets shall be increased to 25% more than the number required for shop-driven rivets. On this basis the shear in field rivets would be 9 600 lb. per sq. in. instead of 9 000 lb.

No information is given in the specifications as to the efficiency of riveted joints. This is an important feature in designing tanks and boilers, as the computations, usually, are made directly on the basis of the efficiency of the joint, rather than by the more indirect process of computing the net section. It would be desirable to include a table showing the computed efficiencies of single- and double-lapped joints and single-, double-, and triple-riveted butt joints for various thicknesses of plates and diameters of rivets. Such tables are used by every manufacturer, and are published by some of them.

The use of the percentage of efficiency in the joint results in a simple formula for determining the thickness of plates in tanks subjected to hydrostatic pressure. Thus, if D = the diameter of the tank, in feet, H = the height, in feet, from the point considered to the surface of the water; the pressure, in pounds per square inch = 0.434 H. The tension in the plate for 1 in. in height = t = 2.6 D H. The required net area of the plate for 1 in. in height = $A = t \div f$, f being the unit stress, in pounds per square inch of net section. For f = 13 000, $A = 2.6\ D\ H \div 13\,000 = 0.0002\ D\ H$. For an efficiency of two-thirds in the riveted joint, the gross area in 1 in. of height = the thickness of the plate, in inches = $A \div \frac{2}{3}$ = 0.0003 D H. To this should be added the additional thickness of $\frac{1}{16}$ in. already referred to. This formula, thickness of plate = 0.0003 D H, makes a convenient and rapid method of determining the thickness of tank plates. For a unit stress in tension of 12 000 lb. the same formula results, if an efficiency of 72% is assumed.

It is customary to design on a basis of efficiency of at least 66%, and that would rule out all single-riveted lap joints. Section 14 specifies lap joints for vertical seams in plates $\frac{3}{8}$ in. thick and less, but does not specify whether they shall be single or double. They should be specified as double-riveted, with the possible exception of $\frac{1}{4}$-in. plates, in which the full strength of the plate is not developed.

Mr Selby. Double-riveted butt joints with the rivet diameters specified in Section 15, will secure an efficiency of at least 70% up to and including a thickness of $\frac{3}{4}$ in. For that reason it would seem desirable to design for an efficiency of 70%, and specify triple-riveted butt joints for plates having a thickness of $\frac{13}{16}$ in. or more.

The unit stress in compression should be limited to a maximum of 15 000 lb. for short columns.

Section 10 is ambiguous in that it allows unit stresses of 20 000 lb. in bracing, without specifying whether the unit stress referred to is for tension, compression, shear, or bearing. It gives a desirable addition to the bracing—which otherwise may be designed too light—to assume an arbitrary initial stress in all bracing members of 5 000 or 10 000 lb.

Section 12, specifying that plates forming the sides of cylindrical tanks shall be of different diameters, is not explicit. Of course, it means that the inside diameter of one plate shall be the same as the outside diameter of the next one.

In Section 13 it is not clear just what is meant by radial seams in a spherical bottom.

Section 16 specifies that plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed, but does not specify the diameter of the punched hole and the amount to be removed by reaming.

Section 29 gives a rule for proportioning the stiffening member around the top of the tank, and specifies that the section modulus may be reduced under certain conditions, but does not specify definitely to what extent it may be reduced.

Section 35 allows an additional height of 6 in. for over-run. This is not clear. Presumably the meaning is that 6 in. in height shall be added to that required for the specified capacity in gallons.

Section 36 specifies that the bracing in towers shall be adjustable. Certainly riveted bracing for towers is better practice, although adjustable bracing may be allowable, on the score of economy, and the riveted bracing should not be excluded.

Section 37 specifies unit stresses in the anchor-bolts not exceeding 15 000 lb. per sq. in. Presumably, this refers to unit tensile strength, but it is not explicit. There is no specification covering the excess of the area at the root of the thread, and no specification which would produce good details in the design of the anchorage connections. Too many anchor-bolts are connected simply through the base plate of the column without provision for developing properly the full strength of the bolt, and this should be covered by a specification which will insure a good detail.

Section 38 specifies concrete sufficient in quantity to take the uplift, but does not provide any factor of safety. It is good practice to provide at least $1\frac{1}{2}$ or 2 times the computed uplift in the weight of the concrete. Furthermore, the specification should include a provision

that the arrangement of the anchorage in the concrete shall be such as to develop the full weight without depending on the tenacity of the concrete to hold the mass together; otherwise, this provision is apt to be overlooked, thus resulting in bad practice. Mr. Selby.

Sections 40 and 41 specify that the quality of materials and workmanship shall be according to certain specifications referred to by name. It would add little to the bulk of these specifications and much to their completeness if the specifications for quality of materials and workmanship were written out in full.

Section 42 specifies a shop coat of approved paint. It may be hopeless to try to get the Engineering Profession to agree on an approved paint, but probably red lead would more nearly satisfy the ideas of most members than any other one kind.

The specification has covered only the work of the designer and manufacturer of tanks, but, inasmuch as most contracts for tanks and stand-pipes, especially for municipalities, include their erection, it would seem desirable to give specifications for erection, testing, and field painting.

It is to be regretted that the author did not see fit to give some text with the specifications.

CHARLES C. HOPKINS, M. AM. SOC. C. E. (by letter).—The author expresses the wish that the general specifications, as submitted in his paper or as finally revised, be adopted by the Engineering Profession. It appears to the writer that, in their present form, the specifications are not drawn as carefully as they should be; that some portions might be omitted, others added, or added to, and other advantageous changes made. Mr. Hopkins.

Although the author evidently intends that there shall be a distinction between a stand-pipe and a tank, he has used the word, "tank," in places where either one is or should be meant, and, in other places, where only an elevated tank can be intended.

It is inferred that by "tower" is meant the framework supporting an elevated tank, that a "tank" may be thus elevated or may stand with its bottom at or near the surface of the ground, and that a "stand-pipe" is simply a tank without a supporting tower, but having a height considerably greater than its diameter.

As the author uses the word "tank" at times in a general sense, including stand-pipe, it leaves considerable uncertainty in other places, for example, in Section 31. If it is assumed that stand-pipes are to be roofed, issue might be taken with the author that they are not generally roofed in the colder climates unless enclosed in a building or "tower," because the floating of the ice which forms around the inside would probably lift, and thus ruin, the roof.

The writer suggests that the weight of water should not be assumed to be 63 lb. per cu. ft., when 62.5 lb. is much more nearly correct and

Mr. Hopkins. a more convenient number to use; or, what would be better still, that Section 3 be stricken out entirely as being unnecessary.

Section 12 appears to be drawn rather carelessly, in several respects. Why refer to the tanks as "cylindrical"? Does not this section refer to stand-pipes also? What is meant by the plates being of different diameters? Possibly the intention is that the courses forming the sides shall lap over each other inside and outside alternately. What is meant by the expression, "No foreign material shall be allowed when caulking"? In specifying that in oil-tank work "both the inside and outside of the tanks shall be beveled for caulking," the author probably means that the inside and outside edges of the plates shall be beveled for caulking; if so, the wording of the section should be changed.

Although the lapping of the side plates alternately inside and outside is the usual construction, what material objection is there to making each horizonal course (especially in small stand-pipes) slightly conical, so that the top of one course enters the bottom of the one next above it? The writer asks this for information, as he has heard a manufacturer of stand-pipes state that he preferred to build in that way, believing it would be more satisfactory, as all the inside caulking of the horizontal joints would be over-handed, and would be done more easily and, perhaps, better.

The author's specifications do not state the manner in which the anchor-bolts are to be fastened to the tank, or the manner in which the bottom of the stand-pipe is to be fastened to the side plates, possibly leaving these points, with others, to be shown on a plan to be submitted, although there is no reference to such a plan in the specifications. As to the connection of the bottom with the side, it is the writer's opinion that for large stand-pipes the use of both inside and outside angles is best, riveting the bottom plates as well as the side plates, to both angles. If either angle is omitted, as it may well be at times, it should be the inner one, because an outside bottom angle gives the stand-pipe a slight additional factor of safety against overturning, and a leak occurring at the angle can be caulked without drawing off the water. Thus far, the writer has always found the outside angle entirely satisfactory.

It is suggested that the specifications should state that all caulking edges shall be planed to a proper bevel, and that all caulking shall be done with a round-nosed tool.

The specifications for the manhole, roof, trap-door, and other details are rather meager, and might perhaps be left out altogether or only made the basis of plans to be submitted by the contractor and to be approved afterward by the engineer. The latter method is not usually as satisfactory as to submit definite plans and specifications on which all contractors may bid.

Mr. Hopkins.

The meaning of Section 35 is not clear to the writer. Certainly several different meanings could be ascribed to it. If it is assumed that the engineer for the "owner" is designing the foundations on which the stand-pipe is to rest, then Section 38 might well be omitted; but if the contractor for the stand-pipe is to build the foundations also, then a complete specification for concrete should be added to Section 38. Unless the contractor for the stand-pipe, tank, or tower, is to build the foundation, Section 11 and Table 2 might as well be omitted, especially the first half of Table 2, because the engineer in charge must satisfy himself as to the safe loads, and for these no general specifications, such as those proposed by the author, should govern.

In reference to painting, the writer believes that such inaccessible parts as "laps in contact" should be either painted or oiled before leaving the shop. Specifications prepared by him and his associates have usually provided that, before leaving the shop, all steel shall be thoroughly cleaned of rust and scale, and be thoroughly coated with boiled linseed oil; that inaccessible surfaces shall be painted before erection; that, after the stand-pipe has been tested and found to be tight, it shall be painted inside and outside with the paint (or varnish) specified; and that no painting shall be done in wet or freezing weather.

Section 16 states that "plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed." The writer believes that all punching of plates should be done from the surface which is to be in contact, and that in plates as thick as $\frac{3}{4}$ in., or thereabouts, the rivet holes should be drilled.

The Manufacturers' Standard Specifications, as revised on February 6th, 1903, are excellent for workmanship, as far as they go, but for stand-pipe and tank work they are not as complete as they should be. The writer would suggest the following, as some of the requirements in workmanship, which are more or less necessary, and are not covered by the Manufacturers' or the author's specifications.

All plates and angles shall be shaped to the proper curve by cold rolling; heating or hammering for straightening or curving shall not be allowed, and no scarfing or other work shall be done on the steel below a specified temperature. The diameter of the die used in punching rivet holes shall not exceed that of the punch by more than $\frac{1}{16}$ in. All rivet holes in horizontal seams of stand-pipes may be punched. All parts shall be adjusted to a perfect fit and shall be marked before leaving the shop. Eccentricity of rivet holes, if not sufficient to cause rejection, shall be corrected by reaming, and rivets of larger size shall be used in the holes thus reamed. The engineer or inspector shall have the right to require driven rivets to be cut out at random in order to inspect the quality of the workmanship. Rivet caulking shall not be allowed, but leaky rivets shall be cut out and replaced with new ones.

Mr. Hopkins. The riveting shall not crack the plates or angles. The plates forming the courses shall be as long as practicable.

Inasmuch as the Manufacturers' Standard Specifications for workmanship are short, it might be well to print them entire, or perhaps only the necessary parts, with any others which might be thought advisable, thus having convenient, for the inspector on erection, the complete specifications governing the work under his charge.

The author's specifications omit mention of the inlet (or outlet) and overflow pipes of stand-pipes, and, although they describe a ladder for a tank, they do not mention one for a stand-pipe—quite a necessary adjunct. It is evident that there should not be an inside ladder in an unenclosed stand-pipe where ice is likely to form.

Assuming that general specifications are needed—of which there may be some doubt—it seems to the writer that, to avoid confusion, it might be best to separate the proposed specifications into specifications for steel stand-pipes and specifications for elevated steel tanks, or to place them under separate headings in the same specifications, following some general specifications of such parts as are common to both.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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A SYSTEM OF COST KEEPING.

Discussion.*

By Messrs. P. L. Reed, W. C. Hammatt, Emile Low, Richard T. Dana, and T. Kennard Thomson.

P. L. Reed, M. Am. Soc. C. E. (by letter). As the author points out, cost records are of greatest value to those who make them and at the time and place at which they are made. Their value diminishes rapidly with time, distance, and changing personnel, for, while it may be permanently recorded that a piece of work cost a definite sum, it becomes increasingly difficult to determine just what that piece of work was, and all that was involved in its construction. The numerator is known, but the denominator is often most uncertain, and the quotient is what is needed as a guide. Records involve the expense of preparation and the mental effort of examination and future reference, and, as costs are amplified, the expense gains on the utility, sometimes to the point where general conclusions, which might be of considerable value, are drowned in masses of details of slight transient importance, which rapidly lose their value until, in practice, the records are not found worthy of the time and mental effort required to consult them. Mr. Reed.

The deviser of a cost-keeping system, which, in some form, is almost essential to the conduct of any business where things are made, must curb his enthusiasm by considerations such as these, and stop before he has gone too far.

* This discussion (of the paper by Myron S. Falk, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Reed. Referring to some of the details of the system described, which seems to be a very good one, the writer occasionally sees a voucher similar to Figs. 1 and 2, on the outside of one fold of which is printed the check upon which the bank makes payment. This seems to have some advantages, as the endorsement on the back of the folded voucher is likewise a receipt for the amount of the bill as summarized on one fold and distributed on the other, much as in the case of that illustrated by the author. The form would also be most convenient for auditing, though, possibly, not so popular with the banks.

The preparation of the time report, Fig. 4, on a fresh card or cards each day, with names, rates of pay, etc., would seem to involve an amount of clerical work out of proportion to its value. If entries are made day by day, the weekly pay-roll form shown in Fig. 9 apparently serves every purpose of that shown in Fig. 4, except to inform the head office daily of the name, hours worked, and rate of pay of each employee, which does not seem very important where the condensed statement of the force account by trades is sent in daily on the form, Fig. 5, and the names, etc., weekly, on the form, Fig. 9.

Where the force is large, or where the work of time and cost keeping is divided between timekeepers and cost clerks, it is necessary to assign numbers or other simple characters to the different sub-heads under which the work is divided, and to enter these characters opposite the names of employees on the time card, so that the labor charges can be properly distributed by one not in close touch with the work on the ground.

Mr. Hammatt. W. C. Hammatt, Assoc. M. Am. Soc. C. E. (by letter).—When in charge of various pieces of construction, the writer has always endeavored to keep close and accurate costs, in order to check, not only the unit costs of different classes by previous work of the same class, but also to present to the company, at the end of the work, an accurate and minutely itemized valuation of the work done, for inventory and insurance purposes. He has used practically the same system in his work as Superintendent of Construction for the Mountain Copper Company, at Martinez, the American Smelting and Refining Company, at Chihuahua, Mexico, and the Mammoth Copper Mining Company, at Kennett, Cal.; therefore, only one of these cases, for example, the construction at Chihuahua, will be described.

The first step of the system involved giving a segregation number to each piece of work. This minimized the clerical work, and also gave convenient reference to ledger accounts. In this system of segregation whole numbers are used for the main divisions of work and decimals of various length for the sub-segregations under these divisions. This provided for a combination of all the decimals of a certain degree when segregations of lesser refinement were necessary for certain purposes. For example:

CONSTRUCTION SCHEDULE. Mr. Hammatt.

General Expense:

01 Superintendence.
02 Surveying and drafting.
03 Office expenses.
04 Proportion, main office.
05 Temporary tracks.
06 Tools, push cars, concrete mixer.
07 Real estate.
08 Warehouse stock.
09 Warehouse expense.

Blast Furnaces:

10 Building.
11 Furnace No. 4, bustle pipe, settler, etc.
12 Changes in present furnaces.
13 Slag and matte equipment.
14 Furnace No. 5, bustle pipe, settler, etc.
15 .....
16 .....
17 .....
18 Connecting blast pipe.
19 Blast pipe in smelter.

(These sub-headings apply to all following headings.)

0.1 Grading.
0.2 Forms.
0.3 Concrete.
0.4 Superstructure.
0.5 Falsework.
0.6 Plumbing.
0.7 Wiring.
0.8 Painting.

Converter Plant:

20 Building (including crane).
21 Converter No. 1 (including stand and tilting system).
22 " No. 2 (" " " " ").
23 Downtakes.
24 Flue.
25 Charge system.
26 Copper disposal.
27 Slag disposal.
28 Blast pipe.
29 Hydraulic system.

Relining System:

30 Building.
31 Relining pit (including tamper and crane).
32 Clay mills.
33 Clay and quartz bins.
34 Track and trestle to same.
35 Tunnel.
36 Drying stands.
37 .....
38 .....
39 .....

Mr. Hammatt. *Charging System:*

40 Tunnel.
41 Trestle.
42 Trackwork.
43 Equipment, charge cars.
44 .....
45 .....
46 .....
47 .....
48 .....
49 .....

Flue System:

50 Dust chamber, steelwork (including hoppers).
51 .....
52 .....
53 .....
54 Tracks.
55 New stack.
56 Equipment, cars, etc.
57 .....
58 .....
59 .....

Power Plant and Blast Mains:

60 North addition (building).
61 South " (").
62 Receiver.
63 Changes in present blowers and connections.
64 Blower No. 4 and connections and motor.
65 " No. 5 " " " "
66 Blowing engine.
67 Blower No. 6, connection and motor.
68 " No. 7, " " "
69 Generating set No. 2.
70 Switch-board.
71 .....
72 .....
73 .....
74 .....
75 .....

Sampling System:

76 Temporary sampling room.
77 .....
78 .....
79 .....

Track System:

80 Standard gauge changes.
81 New standard gauge yard.
82 Slag tracks.
83 Lime track.
84 Track scales.
85 .....
86 .....
87 .....
88 .....
89 .....

0.1 Grading. Mr. Hammatt.
0.2 Track-laying.
0.3 Ballasting.
0.4 Switching.

Shops:

90 Machine-shop building and blacksmith shop.
91 " " plant.
92 Steel " building.
93 Compressor-motor and building.
94 Boiler-shop and round-house building.
95 " " " " " plant.
96 Carpenter shop building.
97 " " plant.
98 Electrical shop building.
99 .....

Bins and Trestles:

100 Main receiving bins.
101 Trestle approach to same.
102 .....
103 .....
104 .....
105 .....
106 .....
107 .....
108 .....
109 .....

Miscellaneous:

110 High-pressure pipe line.
111 Cement sheds.
112 Greek bunk house.
113 Drain tunnel.
114 Smelter stable.
115 .....
116 .....
117 .....
118 .....
119 .....

Mr. Hammatt. *Standard Gauge Equipment:*

120 Oil tank.
121 Locomotives.
122 Wrecking crane.
123 Cars.
124 .....
125 .....
126 .....

In regard to this list, it may be stated that it is faulty in its arrangement, due to the full plans for construction not being complete at the time the work was started. The writer believes in a more complete segregation of fixed and movable plant, for insurance and appraisal purposes, as a machine can be moved to a new site, while its foundation can not, and the superstructure of a building is subject to destruction and damage by fire, while its foundation is not.

AMERICAN SMELTING & REFINING CO.
CHIHUAHUA PLANT

Daily report of labor on_____
for_____
Segregation_____
Date_____

NO.	EMPLOYEES	NO. HRS.	RATE	AMOUNT
	Foreman			
	Carpenters			
	"			
	Helpers			
	Mechanics			
	"			
	"			
	Helpers			
	Stone Masons			
	Brick "			
	Helpers			
	Engineers			
	Firemen			
	Plumbers			
	Helpers			
	Electricians			
	Laborers			
	"			
	Water boys			

FOREMAN

Standard size, 7 by 3¾ in.

FIG. 10.

AMERICAN SMELTING & REFINING CO.
CHIHUAHUA PLANT

Cost report for_____
_____ on_____
Segregation_____
Ending_____

DATE	ACCOUNT	UNITS	RATE	AMOUNT
	Labor			
	do			
	do			
	do			
	do			
	do			
	do			
	Brick			
	Stone			
	Sand			
	Lime			
	Cement			
	Lumber			
	Cost per unit			

Standard size, 7 by 3¾ in.

FIG. 11.

In the matter of construction reports: The forms of two reports used for cost keeping from day to day, are given in Figs. 10 and 11.

That shown by Fig. 10 was made out daily by the foreman for each kind of work done under his supervision, a separate report being made out for each segregation. At the end of the week the form shown by Fig. 11 was made out by the cost clerk from the data on Fig. 10, and the measurement or estimate of each piece of work performed. The form shown by Fig. 10 was also used by the timekeeper as a check on his time, as the total number of man-hours or man-days should agree in both cases, and also the total in dollars and cents for the pay period. Mr. Hammatt

As there is likely to be inaccuracy in progress measurements which are taken daily, or even weekly, it is well to combine the reports of several weeks. These inaccuracies will then disappear, and a truer calculation of unit cost may be made.

This system had to be varied for the different conditions under which the construction work was conducted, for example, in Mexico, where the foremen were mostly illiterate, special men were employed for the cost keeping, and the forms differed slightly from those used in places and on classes of work where men of better mental capacity were employed.

Bills for materials of construction were distributed by the superintendent or cost clerk at the time of checking to the accounts where they belonged. When bills could not be charged to their regular jobs, on account of the actual quantity of material going to each job being unknown, the total was charged to warehouse, and the different jobs debited and warehouse credited as the material was issued.

It may be added that the engineer generally works under difficulties in keeping accurate cost records, as the general idea among American employers is that such data are useless expense, and they will not allow the necessary clerical force to make them accurate and useful. The sentiment among most of them was expressed in the late Mr. T. S. Austin's remark to the writer that whether the brickwork costs \$12 or \$30 per 1 000, the work would go on until it was completed or the company "went broke."

Emile Low, M. Am. Soc. C. E. (by letter).—The system of cost keeping described in Mr. Falk's paper is that used by a contracting company, and is really a compilation of expenses. It shows how the money was expended among different assumed items, and thus affords data for the detection of mismanagement in work of similar nature. Mr. Low.

On construction work it is always interesting for the engineer-in-charge to know, at least approximately, what the work is costing the contractor. For want of absolute knowledge of the cost of some items, it may be difficult to ascertain the total cost, but even a knowledge of the cost of some of the items, as before stated, is of some satisfaction.

It may be stated with some assurance that, during construction,

Mr. Low. nearly every railroad company keeps some kind of a force account, which is rendered to the chief engineer, generally weekly, by his subordinates.

Some years ago, while the writer was in charge of some heavy railroad construction in Southwestern Virginia, he originated and put into effect a system of cost keeping, which showed monthly the cost to the contractor of various classes of work, the statements of which were submitted with the monthly estimates.

The system was extremely simple, and consisted essentially of three blank forms which constituted the basis for keeping the records.

On the railroad in question, each residency consisted of from seven to ten 1-mile sections, thus each residency ranged from 7 to 10 miles in length. The contractor's forces were counted daily by a timekeeper employed by the railroad company, which work comprised his whole duty.

A special timebook was prepared, a double page of which is shown by Fig. 12.

From an inspection of the timebook it will be noted that it is arranged to keep a separate record of each and every piece of work, no matter what it may be. It is also aranged to show the totals for each full week as well as for the partial week. Occasionally, there is a month which includes four full weeks and parts of two weeks, making space for six entries necessary.

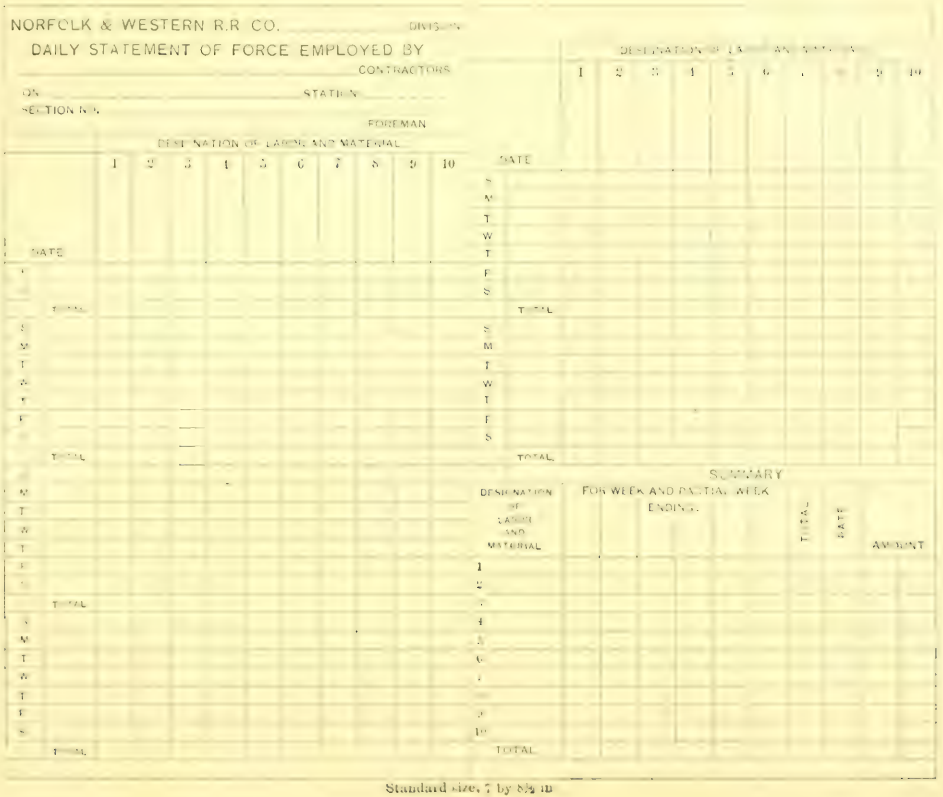
The usual practice is to send in the force accounts weekly to headquarters, and the arrangement of the book facilitates this. At the end of the month there is a summary from which the cost of the work is obtained.

Of course, owing to the method used, the cost obtained is not absolute, as the force employed during the morning and afternoon may vary in numbers, the timekeeper passing over the work only once a day. Where absolute accuracy is required, more timekeepers would have to be employed.

As there are no printed headings, the spaces may be filled in as the nature of the work requires. Space is provided for ten different designations of labor and material, and in cases where there are more items, other pages can be used. It will be seen that the timebook is very flexible, and can be adapted to any contingency.

At the end of the month all work is classified, the form, Fig. 13, being used. The sample sheet shows columns for clearing, grubbing, grading, and masonry. This column is left blank, and the various classes of work are written in, using as many sheets as necessary.

Before the final classification, by sections, the location of each individual piece of work is entered in the column having the printed heading, "Sec." On the grading there may be as many as ten or more forces at work on a mile section, several culverts, bridge abutments, and piers may also be building.


NORFOLK & WESTERN R.R. CO. DIVISION

DAILY STATEMENT OF FORCE EMPLOYED BY CONTRACTORS

ON STATION

SECTION NO. FOREMAN

DESIGNATION OF LABOR AND MATERIAL

DATE	1	2	3	4	5	6	7	8	9	10
S										
M										
T										
W										
T										
F										
S										
TOTAL										
S										
M										
T										
W										
T										
F										
S										
TOTAL										

DESIGNATION OF LABOR AND MATERIAL

DATE	1	2	3	4	5	6	7	8	9	10
S										
M										
T										
W										
T										
F										
S										
TOTAL										
S										
M										
T										
W										
T										
F										
S										
TOTAL										

SUMMARY

DESIGNATION OF LABOR AND MATERIAL	FOR WEEK AND PARTIAL WEEK ENDING							TOTAL	RATE	AMOUNT
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
TOTAL										

Standard size, 7 by 8½ in

FIG. 12.

Mr. Low. The blank form, Fig. 13, may be used in making a compilation showing the cost for separate localities, or the totals for any one section may be entered, these totals being obtained by the addition of the various separate items.

_____ RAILROAD CO.,
COMPARATIVE STATEMENT,
ESTIMATE VS. EXPENSES

RESIDENCY NO. _____ ESTIMATE NO. _____
SECTIONS NOS. _____ _____ DIVISION
MONTH OF _____ _____ CONTRACTOR

	CLASS OF LABOR	RATE	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT
CLEARING	Foremen									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									
GRUBBING	Foremen									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									
GRADING	Foremen									
	Laborers									
	Teams									
	Blacksmiths									
	Carpenters									
	Total Labor									
	Superintendence									
	Explosives									
	Total Expense									
	Estimate									
	Difference									
MASONRY	Foremen									
	Stone Cutters									
	Masons									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									

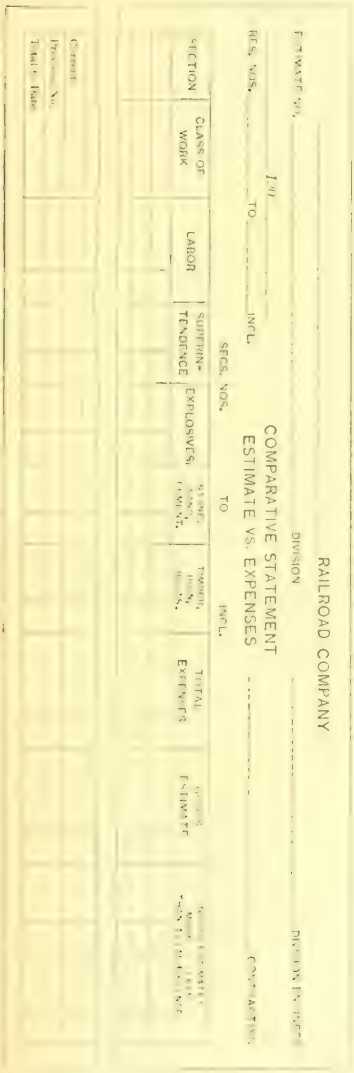
Standard size, 8¼ by 10½ in.

FIG. 13.

Fig. 14 is a blank form showing the summary of all the work on a contract.

The main idea underlying the whole scheme is to make a monthly comparison with the engineer's estimate.

Mr. Low.


RAILROAD COMPANY

ESTIMATE NO. ____ DIVISION ____

RES. NOS. ____ TO ____ INCL. 190

COMPARATIVE STATEMENT

ESTIMATE VS. EXPENSES

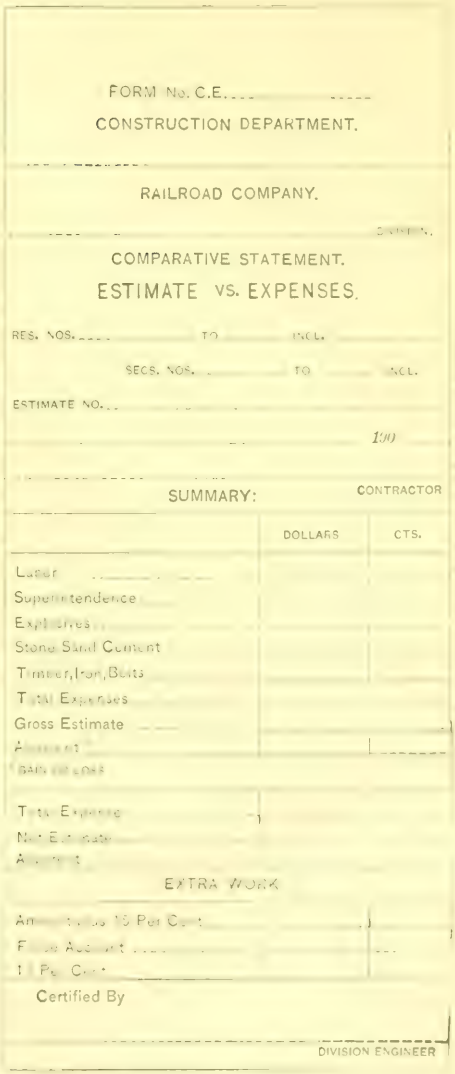
SECS. NOS. ____ TO ____ INCL. CONTRACTOR

SECTION	CLASS OF WORK	LABOR	SUPERINTENDENCE	EXPLOSIVES	STONE, SAND, CEMENT, ETC.	TIMBER, IRON, BOLTS, ETC.	TOTAL EXPENSES	ESTIMATE	GAIN OR LOSS ON ESTIMATE

Current
Previous No.
Total Date

Standard size, 10½ by 8¼ in.

FIG. 14.


FORM No. C.E. ____

CONSTRUCTION DEPARTMENT.

RAILROAD COMPANY.

DIVISION.

COMPARATIVE STATEMENT.

ESTIMATE VS. EXPENSES.

RES. NOS. ____ TO ____ INCL.

SECS. NOS. ____ TO ____ INCL.

ESTIMATE NO. ____

190

SUMMARY: CONTRACTOR

	DOLLARS	CTS.
Labor		
Superintendence		
Explosives		
Stone Sand Cement		
Timber, Iron, Bolts		
Total Expenses		
Gross Estimate		
Amount		
Gain or Loss		
Total Expense		
Net Estimate		
Amount		

EXTRA WORK

Amount due 10 Per Cent		
Force Account		
10 Per Cent		

Certified By

DIVISION ENGINEER

Standard size, 3¾ by 8¼ in. Back of Form shown by Fig. 14.

FIG. 15.

Mr. Low. Referring to the timebook, Fig. 12, there is on a page the cost to the contractor of a particular piece of work, during the month, say, a bridge abutment. An estimate of quantities and cost has been made by the engineer in charge of this particular work.

The details of the cost to the contractor are transferred to the blank form, Fig. 13, the amounts being shown under the heading marked "Sec.," the "estimate" placed under the total, and the difference obtained, which will be more or less as the case may be. These statements then show in detail the expense and estimate of each particular piece of work.

Then a summary is made of each and every kind of work by sections (1 mile), and also a summary of the various estimates. These summaries then show, mile by mile, the status of the various kinds of work.

The engineer's estimate always shows the total cost of a section, and in order to make a proper comparison of all expense, another blank form is needed, which shows a summary of all the labor, superintendence, explosives, stone, sand, cement, timber, iron, bolts, etc., this being the total cost of all the work in a section.

These totals, shown by Fig. 15, may be readily obtained by using the form shown by Fig. 14, for the purpose of tabulation, as the main items are all placed in separate columns, and can be easily added. In some quarters objection is made to keeping an accurate account of the forces employed by the contractor and the tabulated cost, the main reason assigned being that the engineer's estimate is influenced thereby to some extent. This need not be, as the cost to the contractor is not generally known until some time after the estimate is made out. In most public works nowadays there is no classification of excavation, so there are no grounds whatever for manipulation, although there might be under the classification method of dividing excavation into earth, hardpan, and loose and solid rock.

Again, the contractor's costs may be tabulated wholly in the division engineer's office, or even in the office of the chief engineer, he being furnished with duplicate copies of the timebooks, which, as before stated, form the basis of the whole system described.

The writer is a firm believer in keeping costs of contractor's operations, for the very important reason (and almost the only one), that it affords some tangible evidence of fixing suitable unit prices on future work, instead of using that very undesirable makeshift, guessing at them.

Mr. Dana. Richard T. Dana, M. Am. Soc C. E. (by letter).—The author states that "it seems hardly necessary to call attention to the very small value that cost records may have to others than to those who gathered them and who realized and understood fully every local condition at the point of construction." Apparently, he assumes that it is not possible

to record the local conditions with sufficient thoroughness to make such records of value to a man who has not seen the site of the work. Such an assumption involves the further proposition that it is possible to appreciate conditions which it is impossible to express or describe in writing. If the writer is correct in this interpretation of the author's meaning, the author's position is untenable. It is generally accepted that in shop work the conditions can be sufficiently well recorded as to have resulted in the reduction of the art of cutting metals, for example, to almost an exact science.* Mr. Dana.

In out-of-door work there is necessarily interference by the weather, by variation in the conditions of the materials to be handled, by accidents to large units of equipment, and by a considerable number of other causes which do not obtain in the shop. These causes, however, are such as to admit of classification, and it is entirely possible to obtain fairly approximate quantitative expressions for their comparison. The outside field has a great advantage over the shop in that the aid of photographs can be enlisted, and, at an almost infinitesimal cost, photographic records, which are more accurate than the eye and much more durable than the memory, can be obtained, and these records can help out the written notes so greatly as to enable the compilation of practically complete information for future use.

The author's assumption that a man must have seen work done in order to make cost records of it of value to him, means that his memory—his uncertain, hazy, and, perhaps, prejudiced memory—is to be trusted in preference to cold, hard, statistical facts. The writer's experience has been altogether in the opposite direction.

In the third paragraph, the author states that by the use of cost records in the field a contractor can with reasonable accuracy compare the cost of work performed in any specified period with that of any other period. As the local conditions necessarily vary from time to time, it seems difficult to appreciate why the cost from period to period can be compared with benefit, when, according to the author's statement in the preceding paragraph, cost records have small value to others than to those who gathered them, unless the contractor is going to keep the same superintendent on the job from start to finish.

The author has given some blanks showing the method of obtaining cost in the field. It appears as if he had endeavored to combine a cost-keeping system with a bookkeeping system. The bookkeeping system is based on the use of only one unit, the dollar, while cost keeping, as the term has been used in recent years, involves the recording of efficiency, and may utilize any of a considerable number of units. Thus, the cost of drilling may be expressed in minutes per foot; the cost of hauling material may be expressed as the reciprocal of the

* Note the papers by Mr. F. W. Taylor in *Transactions*, Am. Soc. Mech. Engrs., Nos. 1083 and 1119.

Mr. Dana. expression "tons per team-day," and so on. The writer prefers to consider cost keeping as simply a method for recording efficiencies, using whatever units seem most convenient at the time, and to confine it to a class by itself, not allowing it to interfere in any way with the bookkeeper's work, thus avoiding the imposition of a considerable amount of burdensome labor upon the bookkeeper, and using the units most suitable for the case in hand.

Because rates of labor vary in different localities at the same time and at different times in the same locality, the dollar is not even approximately a true measure of efficiency, and the writer prefers to use, as a standard for comparison, the United States Reclamation Service unit of a man-day.

Mr. Thomson. T. Kennard Thomson, M. Am. Soc. C. E.—The speaker is a firm believer in cost analyses, provided all the facts are known, including the personal characteristics of the contractor and his superintendent. For all know that two contractors can take exactly similar contracts under exactly similar conditions as regards labor, material, location, etc., and at the same prices; and while one contractor will make a good profit, the other will sustain a heavy loss, just as one man can start a grocery store or a restaurant, and make his fortune, while another man with the same capital, store, and stock will go bankrupt.

Again, the same contractor's profit or loss will depend largely on his superintendent—unless he acts as his own superintendent. As an example, a well-known contractor had a steam-shovel job with a $5 000 superintendent who left when the job was half through, and a $3 000 man was put in his place. The second man, with exactly the same labor, cars, tools, material, etc., only turned out 60% as much material as his predecessor.

Two of the speaker's clients are contractors—though not competitors—in similar lines of business, and he always advises one to bid 30% higher than the other, and, even then, he feels sure that the first will make a bigger profit than the second.

While cost analysis, therefore, is absolutely necessary, and is much used by the speaker, it would generally be disastrous for a contractor to bid on work in which he was inexperienced, on the strength of the cost analysis made by some one else, and, while the speaker is in favor of cost analysis, he generally wants to know who made it and who will use it.

Vol. XXXV. MAY, 1909. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE MAXIMUM WEIGHTS OF SLOW FREIGHT TRAINS.

Discussion.*

By L. A. Riley, 2d, Esq.

L. A. Riley, 2d, Esq. (by letter).—A close analysis of Mr. Bissell's paper shows some disagreement with observed data, and the writer believes also with theory. The accuracy of Mr. Bissell's weights of trains on compensated grades is not questioned, but they appear to be somewhat low, considering the weight of the locomotive he uses for estimating purposes. Mr. Riley.

The writer's notebook is full of record hauls, but it is not thought that they would serve any useful purpose in this discussion. It might be interesting, however, to give (in Table 4) the tonnage rating of a consolidation-type, slow-freight locomotive, which rating has stood the test of more than a year's operation under all sorts of conditions.

The locomotive in question, as may be seen from Table 4, was considerably lighter than Mr. Bissell's.

That the majority of the figures in Table 4 were obtained in practice the writer can testify, and he can also testify to the fact that at such times no sand was used to increase the adhesion, which was calculated at 25% in making up the rating. The winter rating was for dry track, the only change in calculation being an increase of the rolling friction. In case of icy rails, or stormy weather, it was subject to a further reduction. All of which leads to a disagreement with the 20% adhesion and the theoretical deduction for the same, as given

*This discussion (of the paper by C. S. Bissell, M. Am. Soc. C. E., printed in *Proceedings* for February, 1909) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Riley. by Mr. Bissell. He states that "the tractive power was to be reduced by the proper amount for a constant speed of 10 miles per hour," and, later, that 13.7% was taken as the proper reduction in the tractive power for a speed of 10 miles per hour.

TABLE 4.—FIFTY-CLASS ENGINES; CONSOLIDATION TYPE; CYLINDERS, 22 BY 28 IN.; STEAM PRESSURE, 200 LB.; TOTAL WEIGHT, 144 TONS; WEIGHT ON DRIVERS, 155 156 LB.

Grade.	RATING:	
	Summer.	Winter.
0.38%	2 850	2 485
0.5%	2 424	2 155
0.7%	1 940	1 940
1.0%	1 492	1 386
1.1%	1 386	1 292
1.2%	1 292	1 212
1.4%	1 140	1 078

Compensated actual grades. Speed, 12 miles per hour. Rolling friction, 6 and 8 lb. per ton, respectively. Track in fair condition.

It appears that he calculates the rate of adhesion as 23.2% from the tonnage records, and then reduces this rate by 13.7% for a speed of 10 miles per hour. Sliding friction is subject to a reduction at increased speeds, but the writer does not know where Mr. Bissell finds authority for a reduction in the rate of adhesion with change in speed.

If the rate of adhesion is 23.2% at zero miles per hour, it remains practically the same at any speed. In other words, the tractive power of the locomotive will follow the adhesion from zero speed up to a critical speed at which the locomotive fails to supply steam in sufficient quantities to sustain this pull. This critical speed is different for almost every type and design of locomotive. Fig. 2 may illustrate the point.

Under the best track conditions the locomotive referred to in Fig. 2 holds an even tractive power of 38 800 lb. up to the critical speed of 12½ miles per hour, at which point the capacity of the locomotive boiler reduces the tractive power with increased speed along the curve marked "Cylinder Power at Rim of Drivers."

In regard to freight-train resistances, Mr. Bissell's curve of decrease in resistance due to increase in lading is most interesting, and it would seem that more data might be of value in the matter.

On Fig. 3 the writer has plotted some values, as determined by dynamometer-car tests by Mr. D. C. Crawford in 1901 and by Mr. M. H. Wickhorst in 1904; also a formula proposed by Mr. G. R. Henderson in his "Locomotive Operation," presumably from observed data. These tests were all stated to be for slow speeds and straight

track. Mr. Crawford's and Mr. Wickhorst's figures check remarkably well, and would lead to the conclusion that Mr. Bissell's figures are rather high. Mr. Henderson's formula, however, gives values which come somewhat closer to Mr. Bissell's for heavy loads. The effect of temperature on this curve is not discussed; presumably the values are for conditions above the freezing point. Mr. Raley.

In Equation-2 the question of the definition of tractive power arises. Mr. Bissell calls P the tractive power at the drawbar. From a mathematical and exact point of view, the drawbar pull is a most unsatisfactory figure. The train may be considered as a single body on which a force is impressed, counterbalanced by certain resistances. The force impressed in this case happens to be applied

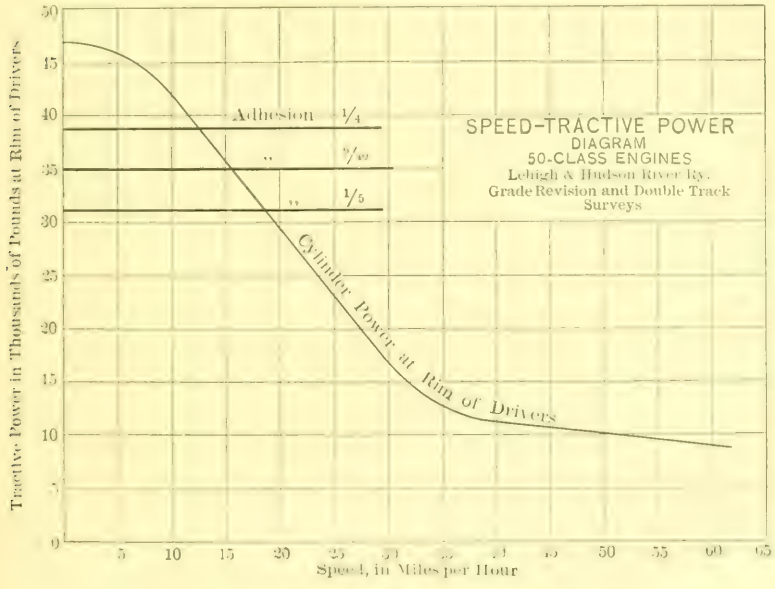


FIG. 2.

by a steam engine mounted on what may be considered one car of this train, but if the engine had been mounted on a concrete base at the summit of the grade, and had exerted its tractive power by a cable, the principle would have been the same. If the engine mounted on the car and the one on the ground were of the same size, design, and steam pressure, the tractive power of each would be the same.

According to Mr. Bissell's formula, however, the tractive power of these two engines would be quite different. There are two balanced forces under consideration, the pull of the prime mover and the resistance of the body moved; the body moved is the total train, not the train back of the engine. The latter consideration will certainly

Mr. Riley. lead to confusion in studying velocity grades. If the train is accelerated or retarded, the drawbar pull, as calculated by Mr. Bissell, is not affected by such acceleration or retardation; but it is obvious that the change in speed does affect the drawbar pull, as the mass of the engine and tender is either absorbing or giving up energy due to its change in momentum. The tractive power of the prime mover, however, is not affected by this change in speed, and the gain or loss in momentum takes its proper place with the other resistances of the body moved.

It has been advanced as an argument against "Cylinder Tractive Power" or "Tractive Power at Rim of Drivers," that an engine with a boiler pressure of 10 or 20 lb. could not move itself, and that it would be ridiculous to assume that it had any tractive power whatsoever. Nevertheless, that engine has just as real a tractive power as the engine which is stalled with its train on a grade. It has a tractive power even though that tractive power is not sufficient to move the mass to which it is applied. What would become of the drawbar pull, as determined in Mr. Bissell's formula, if an attempt were made to apply it to multiple-unit electric trains? One would be simply forced to consider the tractive power of the motors as applied at the wheels and not at the drawbar.

For these reasons it seems to be more logical to modify Mr. Bissell's Equation 2 in some such way as the following:

$$\text{Tons of train weight behind tender} = \frac{T - (20g + l)\,m}{\dfrac{a}{b + w} + 20g}$$

in which

T = Tractive power at rim of drivers,

l = Frictional resistances of locomotive wheels and trucks considered as a loaded car.

The other symbols are the same as Mr. Bissell proposes.

The relation between drawbar pull and tractive power is then as follows:

Drawbar pull, $P = T - (20g + l)\,m$ for constant speed and straight track;

or, to make the formula more universal:

$$P = T - \left[20g + c + l + \frac{1000\,(v_2^2 - v_1^2)}{dg}\right] m$$

in which

c = resistance due to curvature, in pounds per ton of engine weight,

v_1 = velocity, in feet per second, at beginning of period considered,

v_2 = velocity, in feet per second, at end of period considered,

Mr. Riley.

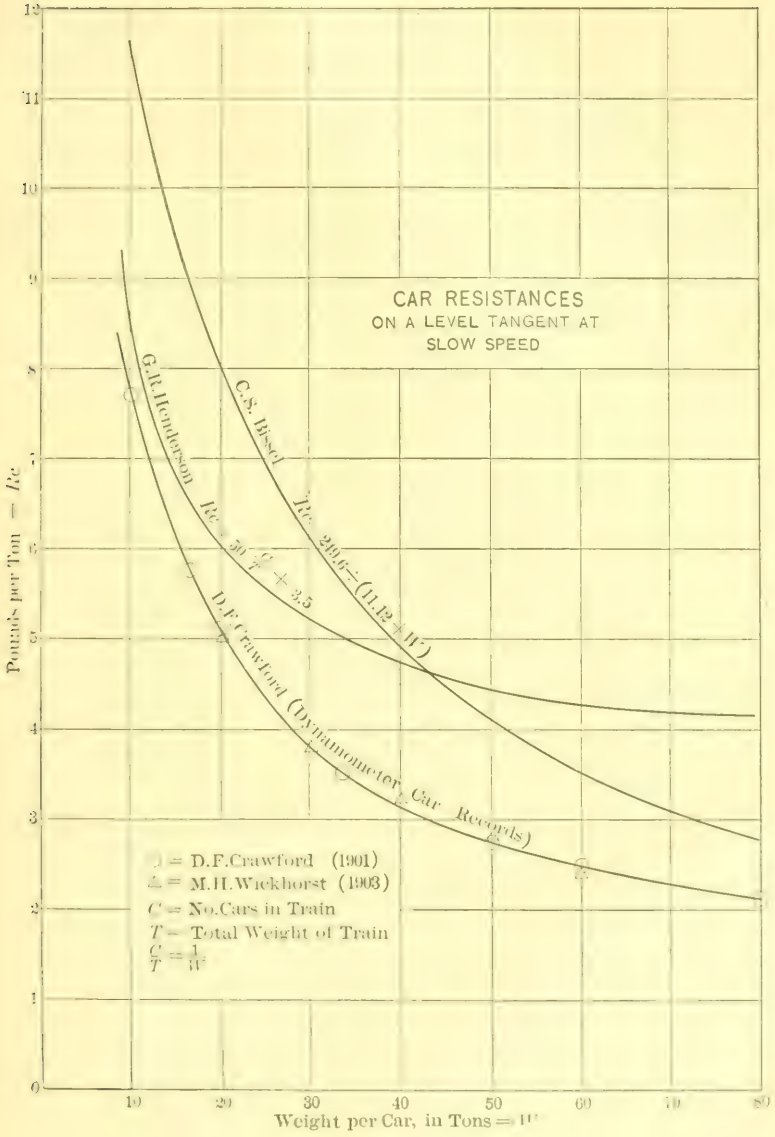


FIG. 3.

Mr. Riley.

d = distance, in feet, covered during period,
m = weight of engine, in tons,
g = acceleration of gravity.

It must be admitted, however, that even this is not exact, since it omits the consideration of the power absorbed or given up by the change in rotative speeds of the engine wheels. However, since it concerns itself only with the engine, it is believed that it is sufficiently exact for all grade-revision purposes excepting possibly the calculation of limiting momentum grades, in which case the methods adopted by Mr. J. B. Berry, in the Union Pacific grade-revision work, are recommended as most exact and logical. A detailed discussion of these methods may be found in the *Proceedings** of the American Railway Engineering and Maintenance-of-Way Association.

* Vol. V.

Vol. XXXV. MAY, 1909. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

NICKEL-STEEL EYE-BARS FOR BLACKWELL'S ISLAND BRIDGE.

Discussion.*

By Gustav Lindenthal, M. Am. Soc. C. E.

Gustav Lindenthal, M. Am. Soc. C. E. (by letter).—The writer desires to add some historical information to this paper. Mr. Lindenthal.

The use of large nickel-steel eye-bars in the construction of the Blackwell's Island Bridge, in New York, originated with the writer when he was Commissioner of Bridges (1902-03).

The cross-sections of the tension members were so large that the use of higher steel for them appeared expedient. Certain tests of nickel-steel eye-bars, 6 to 10 in. wide, made at Pencoyd in 1902, showed that a material some 50% stronger than the usual structural steel, and perfectly reliable for forged work, could be obtained.

At first, eye-bars 18 in. wide and from 2 to 2½ in. thick were intended. The existing eye-bar plants could make bars only up to 16 in., but the large tonnage (17 000 tons) contemplated in the Blackwell's Island cantilever structure and in the chain design for the Manhattan Bridge would have justified the erection of a larger plant for the heavier bars.

The writer was not able to obtain the necessary appropriation for both bridges, but only for the Blackwell's Island Bridge. Because of the smaller tonnage to be contracted for, the largest eye-bars were limited to 16 in., the capacity of the existing plant.

*This discussion (of the paper by William R. Webster, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lindenthal.

The attempts, mentioned by Mr. Webster, of substituting riveted eye-bars for the forged nickel-steel eye-bars, were made after the writer had relinquished charge of the work; so also other changes in the design which resulted in the weak structure, which, from the official investigation and reports made after its completion, Blackwell's Island Bridge (now called Queensboro Bridge) is known to be.

6206

William P. Morse

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXV—No. 6

PRESENTED TO
THE AMERICAN SOCIETY OF CIVIL ENGINEERS
BY
WILLIAM P. MORSE

August 1909

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OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

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AUGUST, 1909.

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NEW YORK 1909

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ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

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TELEPHONE NUMBER.....5913 Columbus.
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INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

May 19th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 137 members and 24 guests.

A paper by C. E. Grunsky, M. Am. Soc. C. E., entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," was presented by the author and illustrated with lantern slides. The Secretary presented written communications on the subject by Messrs. Robert G. Dieck and W. C. Hammatt, and the paper was discussed orally by Messrs. C. E. Gregory, E. Kuichling, and Kenneth Allen.

A paper by W. S. McFetridge, M. Am. Soc. C. E., entitled "Some Extensive Railroad Surveys, and Their Cost per Mile," was presented by the Secretary, and discussed by F. Lavis, M. Am. Soc. C. E., who illustrated his remarks with lantern slides. A written communication on the subject, from Clinton S. Bissell, M. Am. Soc. C. E., was read by the Secretary, and E. W. Lewis, M. Am. Soc. C. E., discussed the subject orally.

The Secretary announced the death of WALTER LORTON DUSENBERRY, elected Junior November 5th, 1890; died May 13th, 1909.

Adjourned.

June 2d, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 136 members and 38 guests.

A paper, entitled "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq., was presented by the Secretary, who also read a communication on the subject from John C. Moses, M. Am. Soc. C. E.

A paper by Henry Japp, M. Am. Soc. C. E., entitled "Caisson Disease and Its Prevention," was presented by the author and illustrated with lantern slides. Written communications on the paper from Dr. J. S. Haldane, and Dr. Fred. L. Keays, were read by the Secretary, and the subject was discussed orally by Messrs. William L. Saunders, Albert J. Loomis, Walton I. Aims, and Seward Erdman.

The Secretary announced the election of the following candidates by the Board of Direction on June 1st, 1909:

AS MEMBERS.

WILLIAM COULSON ARMSTRONG, Chicago, Ill.
BERTRAM BREWER, Waltham, Mass.
ISAAC GLIDDEN CALDERWOOD, Brewerton, N. Y.
AUSTIN BRADSTREET FLETCHER, Boston, Mass.
GEORGE EDWARD HARKNESS, Boston, Mass.
WILLIAM SHERMAN HEWETT, Minneapolis, Minn.
HOWARD MURFREE JONES, Nashville, Tenn.
JOHN BUCK LEONARD, San Francisco, Cal.
WYNN MEREDITH, San Francisco, Cal.
WILLIAM SELBY MITCHELL, St. Louis, Mo.
RICHARD MONROE, Rock Island, Ill.
FREDERICK YANCY PARKER, St. Louis, Mo.
JOHN HOWE PEYTON, Louisville, Ky.
WILLIAM RENTON PHILLIPS, Portland, Ore.
EARLE WILLOUGHBY SAYLES, Watertown, N. Y.
CHRISTOFFER SCHROEDER, Chicago, Ill.

David Gorton Thomas, Denver, Colo.
Julius Edgar Willoughby, Louisville, Ky.

As Associate Members.

Harry Wray Claybaugh, Franklin, Pa.
Bernard Alfred Etcheverry, Berkeley, Cal.
Harley Edgar Frye, Zanesville, Ohio.
Edward Charles Gersbach, Helena, Mont.
Lewis Merritt Gram, Toledo, Ohio.
Harry LeGrand Hilton, Cornwall-on-Hudson, N. Y.
Charles Eugene Johnston, Kansas City, Mo.
Olaf Laurgaard, Conconully, Wash.
William Meier, Chicago, Ill.
John Henry Myers, Harrisburg, Pa.
Samuel Donald Newton, Knoxville, Tenn.
Howard Charles Paddock, Brooklyn, N. Y.
Alan Bigelow Perley, Mt. Airy, Pa.
John Hudson Poole, Washington, D. C.
Henry Blanchard Pratt, Waltham, Mass.
Theodore Eckford Rhoades, New York City.
Edward Henry Rockwell, Tufts College, Mass.
Garret Edward Tilt, Paterson, N. J.
Warren Chamberlain Tudbury, Sanborn, N. Y.
Norman Rand Willard, Douglas, Wyo.
Adolph Yappen, Chicago, Ill.

As Juniors.

Herman Chafetz, Buffalo, N. Y.
Clesson Herbert Field, South Bethlehem, Pa.
Leo Daniel Graham, St. Ignatius, Mont.
Howard Kingsbury Holland, Ann Arbor, Mich.
William James Horrigan, Wilmington, Del.
Frank Ray Howe, Flushing, N. Y.
Franklin Johnson Howes, Rochester, N. Y.
William Richard Hughes, Jr., Monaca, Pa.
Albert August Krieger, Louisville, Ky.
Robert Sherman Larimer, Estes Park, Colo.
Karl Wolfgang Lemcke, Ambridge, Pa.
George Higgins Myers, Denver, Colo.
John Robert Nichols, Brookline, Mass.
Alfred Worcester Nordwell, Cambridge, Mass.
Gilbert Small, Waltham, Mass.
Jonathan Ernest Teal, Lafayette, Ind.
Clement Clarence Williams, Boulder, Colo.
Thomas Judson Wright, Jr., Urbana, Ill.

The Secretary announced the transfer of the following candidates by the Board of Direction on June 1st, 1909:

FROM ASSOCIATE MEMBER TO MEMBER.

JACOB LOUIS BAUER, Elizabeth, N. J.
CHARLES ROBERT BETTES, Far Rockaway, N. Y.
CHARLES CORWIN CREW, Bay City, Mich.
HARRY HARDY, Colon, Panama.
JOHN CLAYTON HOYT, Washington, D. C.
CHARLES GILMAN HYDE, Berkeley, Cal.
CHARLES GILLINGHAM MOORE, Buffalo, N. Y.
CHAUNCY RUSCH PERRY, Waltham, Mass.

FROM JUNIOR TO MEMBER.

JOHN NELSON HAYWARD CORNELL, JR., New York City.

FROM JUNIOR TO ASSOCIATE MEMBER.

CHARLES WHITNEY TILLINGHAST BARKER, JR., Philadelphia, Pa.
HERBERT WATSON HATTON, Wilmington, Del.
GEORGE SAMUEL HILL, San Francisco, Cal.
MATTHEW DETOBIN KELLEY, Lander, Wyo.

Adjourned.

ELECTIONS BY THE BOARD OF DIRECTION, JULY 1st, 1909

At the Business Meeting at the Annual Convention, July 7th, 1909, the Secretary announced the following elections, transfers, and deaths:

AS MEMBERS.

HORACE THOMAS CAMPION, Philadelphia, Pa.
ROBERT ROGERS EVANS, Haverhill, Mass.
WILLIAM HENRY FENN, Wilmington, Del.
JOHN BERTON FERGUSON, Lima, Ohio.
HENRY FOX, Baltimore, Md.
JAMES ADAMS FRENCH, Engle, N. Mex.
ARTHUR GRACE, Medicine Hat, Alta., Canada.
ERWIN ERNEST HASLAM, Niagara Falls, N. Y.
JOHN BENJAMIN HITTELL, Chicago, Ill.
WILLIAM ATLEE JAMES, Kenora, Ont., Canada.
EDWARD NELSON LAKE, Wheaton, Ill.
CHARLES THOMAS MAIN, Boston, Mass.
LAURENCE BRADFORD MANLEY, West Roxbury, Mass.
EDGAR DARWIN MARTIN, Chicago, Ill.
ARTHUR O'BRIEN, Utica, N. Y.
LOGAN WALLER PAGE, Washington, D. C.

George Hamilton Rogers, Tuckahoe, N. Y.
Grant Rohrer, New York City.
Louis Keegan Rourke, Empire, Canal Zone, Panama.
Hugh McGehee Taylor, Mexico City, Mexico.
William Martin Torrance, New York City.
Rudolph Warner Van Norden, San Francisco, Cal.
Friend Pitts Williams, Amsterdam, N. Y.
Philip Brundage Windsor, New York City.

As Associate Members.

William Nicholas Beach, New York City.
Louis Andrew Burns, Rome, N. Y.
Duncan Hugh Campbell, Butte, Mont.
Edward Gilman Carey, New York City.
Clermont C. Covert, Albany, N. Y.
John Charles Davis, Milwaukee, Wis.
Vahram Yettvart Davoud, Provo, Utah.
Thomas Jefferson Edelen, Winnipeg, Man., Canada.
Milton Harvey Freeman, New York City.
Frederick Louis Gagnon, New York City.
Henry Lilburn Gray, Olympia, Wash.
Charles William Hardt, Camp Hill, Pa.
Nicholas Hunter Heck, Key West, Fla.
Clyde Leslie Huff, Salt Lake City, Utah.
Hinman Barrett Hurlbut, New Haven, Conn.
Edwin Gustav Kastenhuber, Jr., Bordentown, N. J.
Howard B. Merrick, Ann Arbor, Mich.
Clair Brandon Patterson, Darrowville, Ohio.
Joel Marsh Pratt, Mobile, Ala.
Ray Howard Rice, Norfolk, Va.
Felix Perceval St. Hill, Hamilton, New Zealand.
Stuart Sims, Moscow, Idaho.
Edward Augustus Southworth, Honolulu, Hawaii.
Harry Clinton Sweeney, Brooklyn, N. Y.
William Edward Thomas, Jamaica, N. Y.
Paul Alphonse Gaillard Tilmont, Nimrod, Mont.
Albert Edwin Waldron, Dallas, Tex.
Jacob Paul Jones Williams, Swissvale, Pa.
Leo P. Winterhalter, Cleveland, Ohio.

As Juniors.

James Everett Beswick, Cold Spring, N. Y.
Arnold Goodwin Chapman, Albany, N. Y.
Pinkney Edward Cunningham, Vicksburg, Miss.
Frank Haskell Densler, Poughkeepsie, N. Y.

Earl Edwin Erdmann, Beverly, Wash.
Harry Carter Gardner, Boulder, Colo.
Gordon Burnett Gifford Hull, Monterey, Mexico.
Stanley Wallace Moore, Manila, Philippine Islands.
Robert Isham Randolph, Chicago, Ill.
Shaler Gordon Smith, Hoboken, N. J.
Adolph Teichert, Jr., Sacramento, Cal.
Ferd. Clark Wintermute, Wilkes-Barre, Pa.

Transfers from Associate Member to Member.

James Simpson Browne, Providence, R. I.
George Heckman Burgess, Albany, N. Y.
Harry Gilbert Burrowes, New York City.
Alvah Benjamin Diehr, Memphis, Tenn.
George Harrison Fenkell, Detroit, Mich.
George Estyn Goodwin, Family, Mont.
Arthur Newhall Johnson, Springfield, Ill.
Francis Mason, New York City.
Hugh Percival Raikes, Birmingham, England.
Francis Libby Sellew, Yuma, Ariz.
Morton Loudon Tower, San Francisco, Cal.

Transfers from Junior to Associate Member.

Elbert Allan Gibbs, Wilkinsburg, Pa.
Walter Joseph Gillen, Brown Station, N. Y.
Thomas Benton Hunter, Monterey, Cal.
Clifford Marshall King, Sandusky, Ohio.
Isaac Solon Matlaw, Schenectady, N. Y.
George Edward Pellissier, New York City.
Carl Howell Reeves, Seattle, Wash.
Horace West Sheley, Salt Lake City, Utah.
Roy Adolf Swartwout, Omaha, Nebr.

Deaths.

Henry Furlong Baldwin, elected Associate Member December 7th, 1892; Member February 6th, 1895; died June 17th, 1909.

Frank Leslie Davis, elected Member September 6th, 1905; died June 9th, 1909.

John Bagley Dimmick, elected Member July 10th, 1907; date of death unknown.

George Samuel Gatchell, elected Member May 7th, 1884; died June 22d, 1909.

William Charles Kernot, elected Member March 6th, 1889; died March 14th, 1909.

FORTY-FIRST ANNUAL CONVENTION
HELD AT BRETTON WOODS, N. H., JULY 6th–9th, 1909.

FIRST SESSION*

Tuesday, July 6th, 1909.—The meeting was called to order at 10 A. M.; John W. Ellis, M. Am. Soc. C. E., Chairman of the Local Committee of Arrangements, in the chair; Chas. Warren Hunt, Secretary; and present, also, about 300 members and guests.

Mr. Ellis introduced his Excellency, Governor Henry B. Quinby, who welcomed the Society to the State of New Hampshire. Onward Bates, President, Am. Soc. C. E., thanked the Governor for his cordial welcome, and then delivered the Annual Address.†

BUSINESS MEETING

Second Session of the Convention, Wednesday, July 7th, 1909.—The meeting was called to order at 8 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 150 members.

The Secretary presented a report of the suggestions received from members as to the place and time for holding the Annual Convention of 1910.‡

On motion, duly seconded, the matter of the selection of the place and time for holding the next Annual Convention was referred to the Board of Direction with power.

The Secretary presented a report of the Board of Direction on the method of appointing the Nominating Committee.§

In reference to the Special Committee on the Status of the Metric System, the Secretary explained that when the committee had presented its report to the Annual Meeting, January 20th, 1909, it had not asked to be discharged, but, through an error, it was discharged. The matter having been brought to the attention of the Board of Direction subsequently, the Board had reappointed the committee.

In the matter of the Progress Report of the Special Committee on Concrete and Reinforced Concrete, the Secretary read discussions by Messrs. John S. Sewell, Clarence W. Noble, and S. Bent Russell,‖ and Messrs. J. F. O'Rourke and J. R. Worcester‖ discussed the report orally.

On motion, duly seconded, it was ordered that the report, and all discussion thereon, together with any reply to the discussion made by the committee, should be received and printed in *Transactions*.

* For the report in full of this and the Business Meetings, see page 276.

† *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 567.

‡ See page 278.

§ See page 282.

‖ These discussions will be printed in the September, 1909, number of *Proceedings*.

The Secretary announced the elections and transfers by the Board of Direction on July 1st, 1909.*

The Secretary announced the death of several members.†

On motion, duly seconded, it was ordered that the programme of the Convention be changed, and that the discussion of professional topics be taken up at 2 P. M., July 8th, and that the Reception and Ball take place in the evening of that day.

Adjourned.

Third Session, Thursday, July 8th, 1909.—The meeting was called to order at 2.45 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 60 members.

Discussion on the topic, "Fire Resistant Construction of Buildings," was opened by a communication from Herbert M. Wilson, M. Am. Soc. C. E., presented by Richard L. Humphrey, M. Am. Soc. C. E. The topic was discussed orally by Messrs. R. W. Lesley, R. L. Humphrey, and S. Whinery.

Discussion on the topic, "Impurities in Sand for Concrete," was opened by Sanford E. Thompson, M. Am. Soc. C. E., and was discussed further by Messrs. R. W. Lesley, C. P. Howard, R. L. Humphrey, G. S. Davison, S. Whinery, C. M. Mills, T. F. Richardson, and Sanford E. Thompson.

The third topic for discussion, "Has Experience Demonstrated that the Oiling of Roads is the Most Satisfactory or Economical Method of Preventing Dust and Preserving the Road Surface?" was presented by S. Whinery, M. Am. Soc. C. E., and was discussed further by G. S. Davison, M. Am. Soc. C. E.

Adjourned.

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* See pages 270–272.

† See page 272.

OF THE BOARD OF DIRECTION

(Abstract)

June 1st, 1909.—Vice-President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Harrison, Kittredge, Knap, Macdonald, Noble, Stuart, Swensson, Thompson, Tillson, and Wilkins.

Ballots for Membership were canvassed, resulting in the election of 18 Members, 21 Associate Members, and 18 Juniors, the transfer of 8 Associate Members to the grade of Member, 1 Junior to the grade of Member, and 4 Juniors to the grade of Associate Member.

A report of the Board to the Society on the method of the appointment of the Nominating Committee was considered.

Director George W. Kittredge was appointed to represent the Society at the Inauguration of Richard Cockburn Maclaurin, as President of the Massachusetts Institute of Technology, to be held June 7th, 1909.

Applications were considered, and other routine business was transacted.

Adjourned.

July 1st, 1909.—A Special Meeting of the Board to canvass ballots for membership, to consider applications for membership, and to consider the matter of the method by which the Nominating Committee is appointed, was held at 8.20 P. M., July 1st, 1909, President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Hazen, Pegram, Stuart, Tillson, and Williams.

Ballots for Membership were canvassed, resulting in the election of 24 Members, 29 Associate Members, and 12 Juniors, and the transfer of 11 Associate Members to the grade of Member, and 9 Juniors to the grade of Associate Member.

A report of the Board in the matter referred to it by the Annual Meeting, concerning the method of the appointment of the Nominating Committee, was adopted.*

Applications were considered.

Adjourned.

July 6th, 1909.—The Board met, as required by the Constitution, at the Annual Convention at Bretton Woods, N. H., at 8.50 P. M., July 6th, 1909, President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Brackett, Churchill, Hodgdon, Horton, Pegram, Stearns, Stuart, Swain, Swensson, Talbot, Thompson, and Williams.

Matters of general interest were discussed.

Adjourned.

* See page 282.

REPORT IN FULL OF THE FIRST SESSION OF THE FORTY-FIRST ANNUAL CONVENTION, AND OF THE BUSINESS MEETING, HELD AT THE MOUNT WASHINGTON HOTEL, BRETTON WOODS, N. H.

FIRST SESSION

Tuesday, July 6th, 1909.—The meeting was called to order at 10 A. M.; John W. Ellis, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, about 300 members and guests.

JOHN W. ELLIS, M. AM. SOC. C. E.—As Chairman of the Local Committee of Arrangements, the duty of calling this convention to order has been assigned to me, and I do now declare that the Annual Convention of the American Society of Civil Engineers is now assembled and ready for business.

The principal function of a local committee is to see that every member and his family enjoy themselves to the utmost extent during the days of the Convention. In order to have your hearty co-operation, the committee would be pleased to have the Secretary, Mr. Hunt, make a few announcements.

THE SECRETARY.—Mr. Chairman, in the printed programme for to-day there is scheduled an excursion through Crawford Notch. The members of the local committee are not only expert in making you happy, but are also weather-wise, and they have decided that this would be a very good day to go to the top of Mount Washington. It has been decided, therefore, to change the programme in that respect, and to let those who wish go to the top of Mount Washington to-day instead of later. The train will accommodate about two hundred, and if there are more than that, the remainder, as well as those who have been up the mountain and do not want to go again, can go on the Crawford Notch trip.

The train for the summit leaves the Boston and Maine station at two o'clock. Tickets should be secured at the hotel office here, and are $3 each for the round trip. The Crawford Notch trip requires that you leave the hotel at 12.45 P. M., and the purchase of a ticket at the hotel, which is 50 cents, and then pay your carriage fare, which is $1, to the carriage agent. The trip to the Crawford Notch is described in the printed programme.

The original programme stated that the Boston Society of Civil Engineers has kindly undertaken to make arrangements for an automobile trip about Boston, for members who may wish to visit that city after the close of the Convention, and, later, they will be glad to furnish facilities to our members to visit any engineering works in the vicinity.

Now, the committee is very desirous of knowing how many wish to take advantage of this offer, and therefore it requests that all such kindly register at the Secretary's office, or give their names to Mr. Dexter Brackett, who is here.

Mr. Ellis.—Mr. President and gentlemen: We have met this year in Annual Convention in a beautiful spot, surrounded by the grand old mountains which are only to be had in the State of New Hampshire, and the American Society of Civil Engineers is to be congratulated that we have with us to-day the Chief Executive of that State. It is with the greatest pleasure, and I consider it a special privilege, that I am able to introduce to you one who some forty-six years ago was associated with me in the acquisition of knowledge in an institution of this State.

He has acquired knowledge; he has had preference accorded to him, and it is my privilege to introduce to you at this time Governor Henry B. Quinby of New Hampshire.

Address of Welcome.

Governor Quinby.—Mr. Chairman and gentlemen: As Chief Executive of New Hampshire, it gives me pleasure to extend a cordial welcome to our State to the American Society of Civil Engineers. I am especially glad to meet you because my father was a member of your profession, and, it is said, an able one.

Our State Engineer is also a member of your Society, and he has made good as an engineer in charge of our State roads, and the last, but not the least, of those three reasons, is that I consider your profession the chief factor in the building up of this great country of ours, and in widening the fame of our nation.

The American engineer recognizes no difficulties too great to be overcome. He not only initiates but he carries through to successful termination; and the achievements of the American engineer are known to the civilized world. I am glad to welcome you here to this particular spot of our beautiful State, surrounded as it is by the magnificent hills in which we take so much pride.

At the entrance to two of these defiles have been placed stone sentinels by the Supreme Engineer of the Universe, and I feel that when your eyes rest especially upon that one guardian at Profile Notch, there will come into your mind the thought that that face has its gaze fixed upon eternity as it stands upon that lofty height, silent and inscrutable; and you will consider, as you gaze, the wonderful events that have occurred since he was placed on guard, ages and ages ago.

But, ladies and gentlemen, I feel it honor enough to meet this distinguished body. I do not feel that I have the right to take up the time which you might better devote to seeing the beauties of these regions. I believe that as you go from here your visit to the White Hills will be forever a pleasant memory, and I certainly wish you, ladies and gentlemen, a most profitable and a most happy occasion.

ONWARD BATES, PRESIDENT, AM. SOC. C. E.—Your Excellency: In the name of the American Society of Civil Engineers, I desire to thank you for your cordial greeting and the tribute which you have paid to our profession. We are now prepared to enjoy ourselves. We have concluded that our Society has made a wise choice in selecting this place for our Convention, and your gracious words have confirmed that conclusion. We are now going to feast our eyes on this beautiful scenery, to breathe this delightful air, to drink your pure mountain water, and to enjoy to the full this wonderful hotel, with all its comforts; and, for the present at least, we shall forget that we will ever have to leave here, but when that inevitable time does come, we shall take away with us pleasant and enduring memories of Bretton Woods.

The President then delivered the Annual Address.*

BUSINESS MEETING

Business Meeting called to order.

Wednesday, July 7th, 1909.—President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 150 members.

THE PRESIDENT.—The meeting will come to order. The first item of business this evening is to consider the time and place for holding the next Annual Convention. I will ask the Secretary to read the suggestions which have been received from members.

Time and Place for Convention of 1910.

THE SECRETARY.—The following is a report of the suggestions received from members of the Society as to the time and place for holding the next Annual Convention:

"As to the place for holding the next Annual Convention:

"Total number of suggestions received, 268, as follows:

Duluth, Minn.	21	Boston, Mass.	7
San Francisco, Cal.	21	Niagara Falls, N. Y.	7
Seattle, Wash.	19	New York City	7
New Orleans, La.	13	Toledo, Ohio	6
Chicago, Ill.	12	Portland, Ore.	6
Washington, D. C.	12	Pittsburg, Pa.	5
Isthmus of Panama	11	Buffalo, N. Y.	5
Detroit, Mich.	10	Mackinac Island, Mich.	4
Los Angeles, Cal.	10	Quebec, Canada	4
Atlantic City, N. J.	10		

"The following places have received three votes each:

Baltimore, Md.	Philadelphia, Pa.
Cape May, N. J.	St. Paul, Minn.
St. Louis, Mo.	Milwaukee, Wis.
Salt Lake City, Utah.	

* *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 567.

"The following places have received two votes each:

Atlanta, Ga.
Denver, Colo.
Havana, Cuba.
Virginia Hot Springs.
Montreal, Canada.
Old Point Comfort, Va.
Rochester, N. Y.
Saratoga Springs, N. Y.
Sault Ste. Marie, Mich.
Indianapolis, Ind.
Minneapolis, Minn.
Ottawa, Canada.

"The following places have received one vote each:

Albany, N. Y.
Asheville, N. C., or some suitable mountain resort in North Carolina.
Bar Harbor, Me.
Boise, Idaho.
Catskill Mts.
Chattanooga, Tenn.
Colorado Springs, Colo.
Delaware Water Gap, Pa.
Fort Smith, Ark.
Galveston, Tex.
Great Falls, Mont.
Honolulu, Hawaii.
Kansas City, Mo.
Lake Mohonk, N. Y.
Mackinaw, Mich.
Madison, Wis.
Mammoth Cave, Ky.
Blue Mt. House, Md.
Narragansett Pier, R. I.
San Juan, Porto Rico.
Spokane, Wash.
Vancouver, Canada.
Yosemite Valley, Cal.
Watch Hill, R. I.
Yellowstone National Park.
Eastern New York State.
Some mountain hotel between Asheville and Lake Erie.
Any place west of Pittsburg that has adequate accommodations.
One of the finished and operating reclamation projects.
Some point in Western Europe.
Grand Cañon of Arizona.
Charter Lake Steamer to visit Cleveland, Detroit, Soo., etc.
Some point on the Great Lakes.

"Time for holding the Convention:

January	1
February	2
March	3
April	7
May (all parts of May are represented in this count)	18
June	71

as follows:

Early June	4
Middle of June	2
Second week in June	2
Third week in June	1
Last week in June	8
Latter part of June	2
Any part of June	52

Time and Place for Convention of 1910.

July 99
as follows:

Early part of July	8
First week in July	6
Second week in July	1
First half of July	2
Middle of July	1
Any part of July	81

August	12
September	9
October	2
November	2

"16 members have no choice.

"The following cannot very well be classified:

June or July	5	February or March	1
June and early July	1	Usual time	1
Latter part June, or 1st of July	1	April or May	2
Early summer	1	Most suitable season	1
During spring	1	September or October	1
May or June	5	Either spring or fall	1
July or August	3	Any time during summer	1
		Cool season	1

THE PRESIDENT.—Gentlemen, you have heard the summary of these votes from members at large. It is for you to take action on them.

JOHN F. O'ROURKE, M. AM. SOC. C. E.—I move that the matter be referred to the Board of Direction.

Motion duly seconded.

THE PRESIDENT.—It has been moved and seconded that this matter be referred to the Board of Direction. Is there any discussion on this motion?

C. H. KEEFER, M. AM. SOC. C. E.—Mr. President, I would like to say that if the Society should be inclined to visit us we should be glad to have them come to Ottawa. I think it is some thirteen years since they last visited there. We should like to have them come to Ottawa, our Capital, being the headquarters of the Government, and the Government Engineers being there. I dare say that there is a great deal that would be of interest to them, and a great deal of information that might be obtained in the matter of the different buildings and public works to be visited. There are many things which would be of interest to the members, and at any time that might be convenient—I do not know whether or not it might be convenient to meet there next year, but if they should come next year or the

year after, or even the year after that, we would be very glad to welcome our American cousins and the members of our Society.

F. P. Stearns, Past-President, Am. Soc. C. E.—Mr. President, I have no suggestion to make in regard to the Convention for 1910. I noticed that eleven members voted for Panama. I did not vote for that place this year, because I thought it was rather early, but I wish to make the suggestion at this time, that there might be a trip there the year after, which would give a great deal of pleasure to those who could go.

The best time to visit Panama is in the spring. That is the dry season. The temperature does not vary from one season to another, but the dry season is altogether the best time to go there.

In order to escape the cold weather here, February or March would be the best month. The work there is enormous, and will be in full swing by 1911. I am not in a position to invite you to go there, but I want to call your attention to the advantages of such a trip. I hear of many engineers who say they intend to go there while this great work is in progress. It is not only the great work that is to be seen, but there is to be seen what the American engineer can do in the organization of a strip of territory in the tropics so that work can be carried on in a thoroughly efficient way.

There is a good hotel there on the Pacific side, at Colon—the Hotel Tivoli. I do not know its capacity, but it will hold a very large number. It has a parlor large enough for a meeting room for an audience of this size, and, in fact, I think that in every way the place is well equipped to receive the members of the Society—as many as would be likely to go.

I do not know what arrangements could be made for the trip, but I think they could be made on reasonable terms, especially if we should be able to use some of Uncle Sam's ships.

The climate is comfortable. There is no danger, I believe, to the health of anyone who goes there, and I think it is something we should look forward to at the proper time.

H. H. Quimby, M. Am. Soc. C. E.—Mr. President, I move an amendment to Mr. O'Rourke's motion. I like the suggestion that has been made that a boat be chartered on the Great Lakes to visit from place to place. I think it would be very interesting and very appropriate. I move, in amendment to Mr. O'Rourke's motion, that a clause be added to it that the Board favorably consider that suggestion.

The President.—Is that motion seconded? There does not appear to be any seconder to that amendment. Is there any further discussion on the original motion?

Arthur Hider, M. Am. Soc. C. E.—Mr. Chairman, it seems to me that it is a long time since there has been a Convention in the South, and I think the members of the Society would enjoy what they might

Time and Place for Convention of 1910.

see there; therefore I would suggest that they consider—and that the Board of Direction consider—very seriously the matter of having the next Convention at New Orleans.

I feel assured that the members, and especially those from this section of the country, would see something that they never have seen, and that would be of interest to them.

Mr. Stearns has suggested going to Panama, but if the trip to Panama could be made next year, instead of in 1911, why, from New Orleans, a party could be arranged to go to Panama.

Of course, a great majority of the members could not afford the time to make the visit to Panama, but some of them could. I hope, gentlemen, that the Board of Direction will consider seriously the matter of having one of the conventions down in the South, and I believe that if they decide to have the next one there, in the early part of the season, in April or May, while everything is frozen up here, it would be a delightful trip to members from this section of the country.

THE PRESIDENT.—Any further suggestions or discussion on this motion? The motion is that the question of choosing the place and time for the next Annual Convention be referred to the Board of Direction. All in favor of this will signify by saying "aye"; contrary, "no." The motion is carried.

Report of Board of Direction on Method of Appointing Nominating Committee.

The next item is a report of the Board of Direction on the method of appointing the Nominating Committee. This will be read by the Secretary.

To the Members of the

AMERICAN SOCIETY OF CIVIL ENGINEERS:

At the Annual Meeting of the Society, January 20th, 1909, the following resolution was adopted:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

The Constitution, Article VII, Section 2, prescribes upon this subject as follows:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting to serve for two years; who, with

the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

"The Board of Direction may prescribe the mode of procedure for appointing this Committee, and fill any vacancies occurring.

"The Committee so appointed shall meet at the Annual Convention of the Society, and nominate candidates to fill the offices, named in Article V, so as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6, and 7.

"A list of nominees for the offices to be filled at the next Annual Election shall be presented by the Committee to the Board of Direction within ten days after the nominees have been selected."

In addition, Article VII, Section 4, says: "At any time before the first day of December, any ten or more Corporate Members may send to the Secretary additional nominations, signed by such members."

The present mode of procedure for appointing the Nominating Committee is as follows:

In the first part of the month of December, suggestion slips are sent out by the Secretary to all the Corporate Members, the slips merely requesting suggestions of suitable persons for members of the Nominating Committee. Said slips are returnable on or before the Annual Meeting following. The suggested names are then tabulated, and the person receiving the largest number of suggestions in a district is usually appointed by the Meeting on a regularly made motion.

After careful consideration the Board of Direction has adopted a new mode of procedure to go into effect this year as follows:

A Preliminary and a Final Suggestion.

1. A preliminary suggestion slip to be sent out on or before September 1st and returnable on or before November 1st.

2. This suggestion slip to contain an explicit request that the corporate membership will not neglect this important duty, and a statement of the great importance of selecting the right men for the Nominating Committee.

3. All the preliminary suggestions received to be tabulated and the names of the three candidates in each district receiving the highest number of suggestions determined.

4. A final suggestion slip to be sent out at least thirty days before the Annual Meeting, as heretofore, giving the three names in each district resulting from the preliminary suggestions; and also stating that any other eligible member of the Society may be suggested.

5. The final suggestion slips to be then dealt with as at present, viz., the result reported to the Annual Meeting.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

July 1st, 1909.

Report on Method of Appointing Nominating Committee.

THE PRESIDENT.—This matter of the method of choosing the Nominating Committee is one that is part of the Board's duty. It was referred to the Board, and, after full consideration, the Board decided to change its method, as has been stated in the report. I do not think that any action can be taken by the members on this report. It is a matter for the Board.

Report of Special Committee on Status of the Metric System.

The next business is the Report of the Special Committeee on the Status of the Metric System. This report was published in the *Proceedings* for October, 1908. Since then we have received a discussion by E. P. North, M. Am. Soc. C. E., who is not present. This discussion can be printed in the *Transactions,* if accepted by the Committee on Publications. It will be in order, therefore, to refer this discussion to that committee. What is your pleasure in the matter, gentlemen?

H. B. SEAMAN, M. AM. SOC. C. E.—Mr. Chairman, is this a final report?

THE SECRETARY.—I think, Mr. President, that some explanation ought to be made. This committee did send in a report, and it was published in the October number of *Proceedings* of last year, and at the Annual Meeting the Secretary understood from the committee that it wished to be discharged, and so stated, and the meeting obligingly discharged the committee. The committee had made no such request in its report, and the Secretary, recognizing his blunder, reported the matter to the Board of Direction, and the Board, using the power vested in it, and using its discretion in the matter, reappointed the committee. So the committee is still in existence, and the report is a progress report.

I do not know that there is any necessity for bringing up this report at this time, but the action of the Annual Meeting was taken without an exact knowledge of the facts, due to the error of the Secretary. The progress report having been published in the *Proceedings,* it does not seem at all necessary to take any action about that, and, if the committee should want to report again, I suppose the matter could come up again.

C. L. STROBEL, M. AM. SOC. C. E.—Is this committee continued indefinitely?

THE SECRETARY.—Yes, sir; the intention, in the appointment of the committee, was that it should be appointed to report from time to time on the "Status of the Metric System."

MR. SEAMAN.—May I ask how many years this is to be continued?

THE SECRETARY.—My recollection is that—I can correct it in the final printing—this committee has been in existence about a year.*

MR. SEAMAN.—How about the previous years?

THE SECRETARY.—It had no previous existence.

* This committee was appointed in January, 1907.—*Secretary.*

MR. STROBEL.—We have had a Committee on the Metric System for many years. It may not have been this committee, but there has been a committee on it. Am I wrong?

THE SECRETARY.—I think you must mean some other society.*

THE PRESIDENT.—Gentlemen, we have the progress report of the Special Committee on Concrete and Reinforced Concrete. This is published in the *Proceedings* for February, 1909, with a statement that "this report has not yet been placed before or accepted by the Society. Its discussion has been made an order of business for the next Annual Convention, July 6th-9th, 1909, and written communications concerning it, received before that date, will be published in *Proceedings*."

Progress Report of Special Committee on Concrete and Reinforced Concrete.

Since then we have received written discussions by John Stephen Sewell, Clarence W. Noble, and S. Bent Russell, Members, Am. Soc. C. E. Have you those discussions here?

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

THE SECRETARY.—Yes, sir, I have them here.

THE PRESIDENT.—Those discussions can be read, if desired. The report is in your hands for action.

G. F. SWAIN, M. AM. SOC. C. E.—Mr. Chairman, I move that these discussions be read, unless they are too long.

MR. O'ROURKE.—Who is going to pass on the length? There is a motion made, and I will second it, that they be read, if they are not too long.

THE PRESIDENT.—The members can decide that as they hear them.

The Secretary read discussions by Messrs. Sewell, Noble, and Russell.†

THE SECRETARY.—That is all the written discussions I have, sir.

THE PRESIDENT.—Gentlemen, this report is before you, for discussion or action.

MR. O'ROURKE.—Mr. President, about twenty-two years ago I used reinforced concrete in enlarging the sides of one of the old piers built for the Poughkeepsie Bridge, to conform to a new design. I wrote a paper on the subject, and it is in the *Transactions* for 1887. Ever since that time I have been using concrete, more or less reinforced. Some of it is being used now in the same manner that buckle-plates are used. It is used to take the place of reinforced units—we will call them—of construction.

Now, I have no faith whatever in reinforced concrete as a unit material. It is a combination, and I object to any report on reinforced concrete, taking it as something like steel or wood, or something that you can depend on, and know that you are getting results which will correspond with data that you have on the subject. There are no two

* A Special Committee to "report a form of memorial to Congress, in furtherance of the adoption of the metric standards * * *," was appointed in February, 1877, and made reports in April and November, 1877. On February 6th, 1878, the following resolution was adopted: "Resolved, that the further consideration of the metric system of weights and measures be indefinitely postponed."—*Secretary.*

† Will be published in the September, 1909, number of *Proceedings.*

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

mixtures of concrete alike, unless each has been skilfully mixed, no matter what the material is, and there has been no proof yet that the large temperature strains which exist on the surface of reinforced concrete, or of any concrete, are going to stand the test of time.

Now, you can have all the report you like, and you need not concur with the minority report, which says that a vote does not make reinforced concrete what you think it is; and all I have to say is that the American Society of Civil Engineers is departing from its old principle of standing by the things that they know, and not depending on the things that they do not know. Now, that is my opinion of those reports.

J. R. Worcester, M. Am. Soc. C. E.—Mr. President, if it is in order, I would like to present a little discussion of the minority report. I presume that is also under consideration.

The President.—Yes, sir; won't you come forward and face the audience, please?

Mr. Worcester read his discussion.*

A Member.—Mr. President, in order to carry this discussion along intelligently, permit me to suggest that you take up the sections of the report for discussion, to prevent any confusion of ideas by going along the entire report.

The President.—I think that would have to come up in the form of a motion. This report has not been accepted by the Society. It is merely published for discussion, as I understand it. The publication is headed: "This report has not yet been placed before or accepted by the Society. Its discussion has been made an order of business for the next Annual Convention." That is, for this meeting. Therefore, it is for you, gentlemen, to determine what you will do with it.

L. J. Johnson, M. Am. Soc. C. E.—Mr. President, it seems pretty clear that this matter has had very careful consideration by a committee which this Society may very well trust.

As I understand it, all that is asked is that this report be accepted precisely the same as the progress reports on the tests of cements have been accepted in the past. It seems to me that nobody supposes that the committee on the testing of cement has yet reached a conclusion not subject to change—to say nothing of the finish of their investigations at the time they reported. It seems, then, as a parallelism with this cement testing, that it is only proper and natural that this carefully thought out body of results should be accepted and put on record where it can be seen.

I do not think anybody will misunderstand them. That is, I do not think serious people will misunderstand them, and reckless people will not be governed by them. Personally, I would not say that I would endorse every conclusion. I do not think I would agree with all

* Will be published in the September, 1909, number of *Proceedings*.

that the committee says on cement testing. I would not feel bound to accept every one of their recommendations, but, as a collection of data embodying the most careful present-day practice in the use of reinforced concrete, it seems to me that the Society would be losing an opportunity by not putting this matter in its permanent archives.

It is very closely akin to the standing recommendations made by the practitioners of Europe, especially the German practice, where I take it that reinforced concrete has been carried to a degree of practicability that this country cannot surpass, if it can be equaled here.

It seems to me that it is very proper that this report be accepted, and I make that motion, Mr. President.

Motion duly seconded.

THE PRESIDENT.—Gentlemen, you have heard the motion. Any discussion upon it? The motion is that the majority report be accepted. A motion is before the house now.

R. W. LESLEY, ASSOC. AM. SOC. C. E.—To be taken as a report of progress?

MR. JOHNSON.—To be taken as a report of progress. I also suggest that the committee be continued, and that the report be published.

THE PRESIDENT.—It has been suggested that it would be better that the motion should read that the report be received and ordered printed in the *Transactions*.

MR. JOHNSON.—That is satisfactory to me.

THE PRESIDENT.—Would you include the discussions that have just been read?

MR. JOHNSON.—I should think so, if the other discussions stand.

THE PRESIDENT.—And the committee to be continued?

MR. JOHNSON.—Yes, sir.

A MEMBER.—Does it include the minority report also?

THE PRESIDENT.—No, sir.

MR. G. F. SWAIN.—Mr. Chairman, I sympathize entirely with the views expressed by Professor Johnson, but I think the minority report ought to be printed, if only as a discussion, in line with the other discussions that have been received as to the majority report of the committee.

THE SECRETARY.—The Secretary also understands that the discussions as submitted should be submitted to the committee, and that the committee should be given an opportunity to write anything it pleases in connection with this original report, in order to make the matter complete in the final report, in the *Transactions*.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—I also suggest that other discussions which come in later follow the same course of procedure that the Secretary has suggested. One of the objects in presenting this report was to draw out discussion which would be helpful to the committee, and I wish to say that I do not believe that any of

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

the discussion which has been heretofore received has been of a constructive character. There has been nothing in the discussions, nor are there any suggestions, in the light of bettering the report of the committee. The discussions which the committee particularly desired to have are those which would suggest improvements in the report itself, and I believe the purpose will be served if the discussions, as they come in, be referred to the committee and published.

I would like to say, also, that there are some typographical errors in the report, and these the committee would like to have corrected; for instance, there is one in which the angle, the bent-up angle, reads 75°, whereas it should read, 45°, just as Captain Sewell suggested.

It may be a matter of interest to state that this committee was by resolution asked to co-operate with Committees of the Maintenance of Way Association, and of the Society for Testing Materials, and it might be interesting to know that of the total membership of that committee—some thirty-three—twenty-seven were for the report, four were against it, and two did not vote; and, of the twenty-seven members who voted for the report, twenty-five were members of this Society.

THE PRESIDENT.—Are you ready for the question? Those in favor of the motion will signify it by saying "aye"; contrary, "no." The motion is carried. The Secretary has a paper which he desires to read.

Letter from San Francisco Association, Am. Soc. C. E.

THE SECRETARY.—The Secretary forgot to read the following letter, when the matter of the time and place for the Annual Convention was up:

"SAN FRANCISCO, CAL., June 25, 1909.

"CHARLES WARREN HUNT, Secretary,
"Am. Soc. C. E.,
"220 W. 57th St.,
"New York City.

"DEAR SIR:

"At the regular bi-monthly meeting of the San Francisco Association of Members, the matter of the Annual Convention of 1910 was discussed at length, and it was unanimously resolved that—inasmuch as the Pacific Coast, and particularly this City, as the geographical, industrial, and financial center thereof, appears to present superior attractions for such an occasion in the way of important engineering works, and evidence of rapid material development; and, inasmuch as San Francisco, America's eighth city in population and wealth, famed the world over for its generous and well-ordered hospitality, has already, three years after utter physical ruin, eclipsed its former self with larger and finer buildings and better and cleaner streets, and will by next year present facilities for varied entertainment unrivaled by those of any other place—the Society should be cordially invited to hold its Annual Convention in this City in the autumn of 1910. In extending this invitation, we respectfully invite your attention to the following works, constituting a few of the attractive features that would attend an excursion of engineers to San Francisco:

"In San Francisco and its immediate suburbs, in addition to the latest types and methods of building construction and equipment, and the latest developments in connection with public service, are to be seen the latest improvements in dock construction, the immense dry-dock and ship-building plants at Mare Island and Hunter's Point, and the Union Iron Works (where the U. S. Battleship *Oregon* was built), the electrification of the extensive suburban lines of the Southern Pacific Company, the electric power plants at Martin Station, South San Francisco, with its three 5 400-h.p. gas-engine units, and, at Oakland, where a single Curtis turbine unit of 9 000-kw. capacity is in successful operation.

"Short excursions may be arranged to include with convenience visits to the following points of interest: The plants of the Pacific Gas and Electric Company at Electra, with a capacity of 2 000 kw., under 1 460-ft. head; at de Sabla, with 14 000-kw. capacity under 1 530-ft. head; at Centerville with a 5 500-kw. Francis turbine operating under 590-ft. head (the highest in the world); the high-tension (60 000 volts) circuits from Electra and de Sabla are, respectively, 140 and 250 miles long;

"The plant of the Great Western Power Company near Oroville, a 60 000-h.p. installation with about 160 miles of steel towers carrying two transmission circuits with a capacity of 100 000 volts each;

"Gold-dredging work near Oroville and Folsom with twenty-four dredgers representing an investment of $7 500 000.00 in machinery, working gravel which produces 6 to 10 cents per cubic yard at as high a rate as 300 000 cu. yd. per month for the largest dredgers at a cost of less than 2 cents per cubic yard, the annual output being $10 000 000.00;

"Smelting works at Selby's, near San Francisco; the copper smelters at Kennett and Coram, and the celebrated electric furnace at Heroult in Shasta County;

"Three large Portland cement plants within fifty miles of San Francisco; the immense petroleum and asphalt industries of Kern, Fresno, Santa Barbara, Los Angeles, and Orange Counties.

"Farther removed from San Francisco, but easily accessible en route, may be mentioned the Owens River Aqueduct construction work in Southern California, the Klamath and Orland irrigation works in Northern California, and the Yuma and Roosevelt Dams.

"San Francisco is the center of the many beautiful pleasure and scenic resorts along the coast and in the mountains of California. These points of interest are so well-known that further reference to them need not be made.

"San Francisco's equipment, both natural and artificial, for entertaining guests is certainly unexcelled; the weather is fair and balmy, twenty-four hours of every day, during the Autumn months, being a comfort and delight. Among scores of good-sized, first-class hostelries and many brilliant cafés, this City has three of the largest and most modern hotels in the world, steel frame, fire-proof construction, the best of service, and very moderate rates. The many theaters are of beautiful architecture, steel-frame, fire- and earthquake-proof construction.

"We respectfully submit that in point of time it is highly

Letter from San Francisco Association, Am. Soc. C. E.

opportune that the Society favor San Francisco with a visit, and we can assure the members that they will have set before them such a feast of reason and observation and flow of soul that they will of one accord declare that convention to be the most successful ever held by the Society.

"We send you under separate cover, with our compliments, birds-eye views of San Francisco, taken shortly after the earthquake and early this year, respectively, which will afford some idea of what has been done to reclaim the vast area of ashes and ruins that resulted from the fire of 1906. We also send you a small map of California on which are indicated the locations of some of the engineering works of interest in this vicinity.

"Trusting the one hundred and ten members of our Association, together with the balance of the three-quarters of a million people residing in San Francisco and its immediate suburbs will have the pleasure of entertaining the Society during the Autumn of 1910, we are,

"Sincerely yours,
"San Francisco Association of Members, Am. Soc. C. E.
"LUTHER WAGONER,
"President.

"E. L. THURSTON, JR.,
"Secretary."

Letters from Mayor and Citizens' Business League of Milwaukee.

I also have an invitation from the Mayor of Milwaukee, and from the Citizens' Business League of Milwaukee, to hold the next Convention in that city.

Announcements.

I also have to announce, in the regular order, the following elections and transfers by the Board of Direction on the 1st of July.*

It may be interesting to state that there are eighty-five names on this list, and that they come from twenty-nine of the subdivisions of the United States, three from the subdivisions of Canada, two from Mexico, one from New Zealand, one from England, one from Panama, and one from the Philippine Islands.

I also have to announce, Mr. President, with regret, the following deaths.†

The programme says, Mr. President, that to-morrow evening there will be a meeting in this hall for the discussion of professional topics, which have been suggested.

Discussion on Change of Programme.

MR. O'ROURKE.—Mr. Chairman, I was approached by a number of ladies, and several of their admirers, and requested to say that it would please them very much if we would postpone the topics until Friday night, and have the ball to-morrow night, instead of Friday night. So I move that we substitute for the discussion of topics to-morrow night the programme that was arranged for Friday night, the Reception and Ball, having on Friday night that which was intended for to-morrow night.

* See pages 270-272.

† See page 272.

Motion duly seconded by Mr. Hatton.

THE PRESIDENT.—It is before the house, and open for discussion.

MR. WORCESTER.—Mr. President, it seems to me that that is doing great justice to the ladies and their admirers, but hardly fair justice to the members of the Society who have interest in the professional topics of the Society. I think that a great many have made their plans to attend the meetings of the Convention, and have not made plans to stop over for the social portion on Friday, and I think it would be hardly fair to them to put them off in this way.

S. WHINERY, M. AM. SOC. C. E.—Mr. Chairman, would there be any objection to holding a meeting for this discussion to-morrow afternoon, in order to accommodate the members who wish to go away early? I merely ask for information.

THE PRESIDENT.—I think the meeting can determine that.

MR. O'ROURKE.—Mr. Chairman, talking to the motion, I think that these conventions are intended for two things: first, the discussion of the same things that we are discussing all the year, and next the meeting with one another which we get once a year. Now, a great many people are going on Friday, and will not be here on Friday night, and it is all right to say that you can discuss, and you can work, and you can grind, but it is said that Jack is a dull boy unless he gets a little play once in awhile, and I think this is a case of that kind; therefore, in justice to our folks who come here to enjoy themselves, I think we ought to adjourn our business meeting and give the ladies their innings to-morrow night. I do not see what harm there is in having that Reception to-morrow night instead of waiting until most of the people have gone home; anyway, this is the first Convention at which I ever knew the Ball to be postponed until the last night, and therefore, as the members can arrange to discuss anything they like at any time they wish, I think, in justice to those who are not members themselves, but are the members' bosses, that we should let them have their innings to-morrow night.

THE SECRETARY.—Mr. President, would there be any objection to having this meeting for the discussion of professional topics to-morrow afternoon?

A MEMBER.—No.

MR. SEAMAN.—To-morrow morning?

THE SECRETARY.—That was not my question.

MR. SEAMAN.—I think that we have been very seriously criticized abroad because we do not have as much professional discussion as we had in former years, and, while I have the deepest sympathy for Mr. O'Rourke's proposition and would be glad to vote for it, I think we should let other things step aside for professional discussion. We have had all day given up to enjoyment, and plan to give all day to enjoyment to-morrow, and now we are asked to give up to-morrow evening

Discussion on Change of Programme.

for enjoyment, and still no discussion. I should like to have the discussion to-morrow afternoon, if it cannot take place to-morrow morning.

THE SECRETARY.—There is no reason why it cannot take place all day, so far as I know.

MR. SEAMAN.—That is what we ought to have. I do not think we have enough of it.

MR. WHINERY.—May I ask further—the Secretary is better informed than most of us about the programme, on both social and business matters—what time to-morrow would be most convenient, with reference both to pleasure and to business, to hold this session, if one is decided on?

THE SECRETARY.—I do not think it makes any very great difference, Mr. President, but it seems to me that the afternoon would be the better time. There is nothing to interfere with it in the afternoon.

THE PRESIDENT.—What hour would you suggest?

THE SECRETARY.—I should say two o'clock.

MR. WHINERY.—In order to bring it before the meeting, I move that a meeting of the Convention for the discussion of engineering topics be held at two o'clock to-morrow afternoon.

THE PRESIDENT.—Mr. Whinery, there is a motion before the house.

MR. O'ROURKE.—I accept the amendment, Mr. President.

THE PRESIDENT.—The motion then, as I understand it, is that, instead of holding the meeting for the discussion of professional topics to-morrow evening at eight o'clock, it will be held at two o'clock to-morrow afternoon, and that the time for the Convention Ball will be changed from Friday evening to Thursday evening. Are you ready for the question? Those in favor of the motion will signify it by saying "aye"; contrary, "no." The motion is carried.

Announcement.

THE SECRETARY.—I am reminded that I should announce to you that the certificates for the purchase of your return tickets are ready for you, having been properly viséd by the railroad man; they can be obtained at the Secretary's office at any time.

THE PRESIDENT.—This ends all the business for the evening. A motion to adjourn will be in order.

Motion to adjourn duly seconded; unanimously carried.

Adjourned.

FORTY-FIRST ANNUAL CONVENTION

EXCURSIONS AND ENTERTAINMENTS

Excursions had been arranged by the Local Committee to the Summit of Mt. Washington, to Crawford Notch, and Bemis, and to the Profile House. Owing to weather conditions there was a slight change in the programme, and the large party which went to the Summit on the afternoon of July 6th were fortunate in securing a clear view. Nearly every one present made the trips above mentioned, and many also visited "The Flume" and drove to the Summit of Mt. Willard. These trips, together with the delightful short walks and drives, were much enjoyed.

A well-attended and enjoyable Reception, with dancing, was held on the evening of Thursday, July 8th

THE ATTENDANCE AT THE FORTY-FIRST ANNUAL CONVENTION

The following 177 members were in attendance. There were also present 145 ladies and others of the families of members.

Aiken, W. A..Philadelphia, Pa.
Allen, C. Frank...Boston, Mass.
Allen, C. M....Worcester, Mass.
Allen, Kenneth..New York City
Alvord, J. W.....Chicago, Ill.
Archbald, James..Scranton, Pa.
Arnold, W. H..New York City
Atwood, W. G.Indianapolis, Ind.
Auryansen, F..New York City

Babcock, W. S..New York City
Baldwin, F. H..Bayonne, N. J.
Ball, C. B.....Chicago, Ill.
Barbour, F. A.....Boston, Mass.
Bates, Onward.....Chicago, Ill.
Bennett, L. J....Buffalo, N. Y.
Bensel, J. A....New York City
Beugler, E. J....New York City
Bott, J. B.....Greensburg, Pa.
Bowers, George...Lowell, Mass.
Brackett, Dexter..Boston, Mass.
Breed, C. B.....Lynn, Mass.
Brewer, Bertram.Waltham, Mass.
Brooks, Fred.....Boston, Mass.
Brooks, J. P.....Urbana, Ill.
Brown, W. P.,
South Orange, N. J.
Burden, James.....Troy, N. Y.

Carpenter, C. E..Yonkers, N. Y.
Chase, R. D..New Bedford, Mass.
Churchill, C. S....Roanoke, Va.
Clapp, O. F..Providence, R. I.
Codwise, E. B..Kingston, N. Y.
Conkling, C. C..Buffalo, N. Y.
Connor, E. H.,
Leavenworth, Kans.
Cook, J. H.....Passaic, N. J.
Cotton, J. P....Newport, R. I.
Craven, Alfred..Yonkers, N. Y.
Crocker, H. S.....Denver, Colo.
Cummings, R. A..Pittsburg, Pa.
Cunningham, A. O.,
St. Louis, Mo.

Davis, C. E....Kingston, N. Y.
Davis, J. L.....New York City
Davis, W. R.....Albany, N. Y.
Davison, G. S....Pittsburg, Pa.
Dean, A. W....Nashua, N. H.
Dean, Luther....Taunton, Mass.
Doten, L. S..Washington, D. C.
Douglas, B....Grosse Isle, Mich.
Dutton, C. H..Waltham, Mass.

Elliott, M..Culebra, Canal Zone
Ellis, J. W...Woonsocket, R. I.

Eltinge, O. L..Kingston, N. Y.
Emerson, G. C....Boston, Mass.

Farnum, L. N..New York City
Flad, Edward...St. Louis, Mo.
Follett, W. W....El Paso, Tex.
Ford, W. G....Brooklyn, N. Y.
Foss, F. E.....Pittsburg, Pa.
Francis, G. B...New York City
French, A. H...Brookline, Mass.

Gardiner, F. W..Yonkers, N. Y.
Gaston, L. P..Somerville, N. J.
Gerber, Emil.....Pittsburg, Pa.
Green, B. R..Washington, D. C.
Greiner, J. E....Baltimore, Md.

Haines, H. S...New York City
Hale, R. A....Lawrence, Mass.
Hammond, G. T.New York City
Hatton, T. C..Wilmington, Del.
Hazen, J. V.....Hanover, N. H.
Henderson, J. T.,
Hartford, Conn.
Herbert, H. M.,
Bound Brook, N. J.
Hider, Arthur.Greenville, Miss.
Hill, W. R.....Albany, N. Y.
Hodgdon, F. W...Boston, Mass.
Hogue, Chester J..Boston, Mass.
Holden, C. A...Hanover, N. H.
Horton, Horace E..Chicago, Ill.
Howard, C. P..Cleveland, Ohio
Humphrey, R. L.,
Philadelphia, Pa.
Hunt, Charles Warren,
New York City
Hyde, C. G.....Berkeley, Cal.

Johnson, L. J.Cambridge, Mass.
Johnson, T. H....Pittsburg, Pa.
Johnson, W. S....Boston, Mass.
Jordan, E. C.....Portland, Me.

Keefer, C. H.,
Ottawa, Ont., Canada
Kimball, G. A....Boston, Mass.
King, W., Jr...New York City
Knickerbacker, John,
Troy, N. Y.
Koon, R. E...Kansas City, Mo.

Lesley, R. W..Philadelphia, Pa.
Lewis, E. W..New Haven, Conn.
Lewis, N. P....Brooklyn, N. Y.
Lindenthal, G..New York City
Loomis, Horace..New York City
Loweth, C. F.....Chicago, Ill.
Lowinson, Oscar.New York City
Lynch, M. L.....Tyler, Tex.

McKenzie, T. H.,
Southington, Conn.
Manley, Henry....Boston, Mass.
Manley, L. B.....Boston, Mass.
Miller, C. H..Little Rock, Ark.
Miller, R. P...New York City
Mills, C. M....Philadelphia, Pa.
Morrow, J. J.,
Washington, D. C.
Morse, W. P....Newton, Mass.
Moses, J. C..Cambridge, Mass.
Mott, W. E....Brookline, Mass.
Murtaugh, M. M.,
Salt Lake City, Utah

Ockerson, J. A..St. Louis, Mo.
O'Rourke, J. F..New York City
Ostrander, J. E..Amherst, Mass.
Owen, James..Montclair, N. J.
Owen, K. D...Montclair, N. J.

Pegram, G. H..New York City
Perry, F. W....Brooklyn, N. Y.
Polk, W. A.....New York City
Powers, J. A.....Troy, N. Y.
Putnam, G. R.,
Washington, D. C.

Quimby, H. H.Philadelphia, Pa.
Quincy, C. F...New York City

Richardson, T. F.,
Cambridge, Mass.
Ricker, G. A....Buffalo, N. Y.

Ricketts, P. C.....Troy, N. Y.
Ridgway, R.,
Poughkeepsie, N. Y.
Riggs, M. J.....Toledo, Ohio
Rollins, J. W., Jr.,
Boston, Mass.

Sax, P. M....Philadelphia, Pa.
Scammell, J. K.,
St. John, N. B., Canada
Seaman, H. B..New York City
Skinner, F. W..New York City
Skinner, J. F..Rochester, N. Y.
Small, G.....Waltham, Mass.
Smith, E. M...New York City
Smith, J. Waldo..New York City
Smith, M. H.,
White Plains, N. Y.
Spear, W. E....Brooklyn, N. Y.
Spofford, C. M..New York City
Staniford, C. W.,
Brooklyn, N. Y.
Stearns, F. P....Boston, Mass.
Stearns, R. H..New York City
Stowe, H. C....Brooklyn, N. Y.
Strobel, C. L.....Chicago, Ill.
Stuart, Francis Lee,
Essex Fells, N. J.
Sutton, Frank,
Washington, D. C.
Swain, G. F.....Boston, Mass.
Swensson, Emil...Pittsburg, Pa.

Talbot, A. N.....Urbana, Ill.
Thackray, G. E..Johnstown, Pa.
Thomas, C. D..Brooklyn, N. Y.
Thompson, F.....Boston, Mass.
Thompson, S. C..New York City
Thompson, S. E.,
Newton Highlands, Mass.
Thorn, A. W....Buffalo, N. Y.
Tighe, J. L.....Holyoke, Mass.
Tinkham, S. E....Boston, Mass.
Tripp, O. H.....Rockland, Me.

Van Ornum, J. L.St. Louis, Mo.
Verveer, E. L..New York City

Wagner, J. C..Philadelphia, Pa.
Watt, D. A.....Albany, N. Y.
Webster, G. S..Philadelphia, Pa.
Wendt, E. F.....Pittsburg, Pa.
Wentworth, C. C..Roanoke, Va.
Weston, R. S..Brockton, Mass.
Whinery, S..East Orange, N. J.
White, J. G....New York City
White, L.....New York City
Whitney, F. O....Boston, Mass.
Williams, G. S.,
Ann Arbor, Mich.
Wilson, P. H..Philadelphia, Pa.
Wing, C. B.,
Stanford University, Cal.
Woodbury, C. J. H.,
Boston, Mass.
Woodruff, E. L.,
Los Angeles, Cal.
Woods, H. D.,
West Newton, Mass.
Worcester, J. R..Boston, Mass.
Wyckoff, C. R., Jr.,
Brooklyn, N. Y.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

September 1st, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Concrete Piles," by Howard J. Cole, M. Am. Soc. C. E., and "Hydro-Electric Power in Canada," by Cecil B. Smith, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for May, 1909.

September 15th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Use and Care of the Current Meter, as Practiced by the United States Geological Survey," by John C. Hoyt, Assoc. M. Am. Soc. C. E., and "Walnut Lane Bridge, Philadelphia," by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

October 6th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "A Review of Chicago Paving Practice," by P. E. Green, Assoc. M. Am. Soc. C. E., and "The Purification of the Water Supply of Steelton, Pennsylvania," by James H. Fuertes, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

October 20th, 1909.—8.30 P. M.—A paper by Chester W. Larner, Esq., on "Characteristics of Modern Hydraulic Turbines," will be presented for discussion.

This paper is printed in this number of *Proceedings.*

PAPERS AND DISCUSSIONS

The first and second volumes of *Transactions* for 1909 (Volumes LXII and LXIII) have been issued. Two additional volumes will be issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings.* The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the third and fourth volumes for 1909 will be issued on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

(Abstract of Minutes of Meeting)

June 18th, 1909.—The twenty-fifth regular meeting of the San Francisco Association of Members, Am. Soc. C. E., was held; Luther Wagoner, President, in the chair; E. T. Thurston, Jr., Secretary; and present, also, 35 members and guests.

It was voted to hold the weekly informal luncheons hereafter at 12.15 P. M. every Wednesday, at the Bismarck Café.

The Secretary read a communication from the Secretary of the Society calling attention to the fact that the publication of discussions in the *Proceedings* of the Association would prevent their subsequent publication in the *Transactions* of the Society, and suggesting the advantage of forwarding discussions directly to the Society, thereby insuring greater publicity. It was agreed that such would be the procedure in the future.

* *Proceedings*, Vol. XXXIII, p. 30 (January, 1907).

The Secretary reported the acquisition of three new members since the last meeting.

Mr. William George, manager of a local cement factory, was introduced as one of the guests of the evening, and invited the Association to visit the works of his company. The invitation was accepted, and the President and Secretary were directed to represent the Association in making the necessary arrangements.

The matter of the Annual Convention of the Society for 1910 was discussed, and a motion to invite the Society to hold its next Convention in San Francisco was carried unanimously.

The paper by W. S. McFetridge, M. Am. Soc. C. E., entitled "Some Extensive Railroad Surveys, and Their Cost per Mile," was discussed by Messrs. G. L. Dillman, W. C. Hammatt, and F. Riffle.

The paper by Howard J. Cole, M. Am. Soc. C. E., entitled "Concrete Piles," was discussed by Messrs. L. J. Mensch, M. L. Tower, J. H. Wise, and S. G. Hindes.

The paper by D. A. Usina, Assoc. Am. Soc. C. E., entitled "Copyright in Drawings of a Technical Character," was discussed by L. J. Mensch, M. Am. Soc. C. E.

Adjourned.

ACCESSIONS TO THE LIBRARY

(From May 11th to July 31st, 1909)

DONATIONS*

POCKET-BOOK OF USEFUL FORMULÆ AND MEMORANDA FOR CIVIL, MECHANICAL AND ELECTRICAL ENGINEERS.

By *Sir* Guilford L. Molesworth and Henry Bridges Molesworth. Twenty-Sixth Edition, Revised and Enlarged with an Electrical Supplement by Walter H. Molesworth. Leather, 5 x 3 in., illus., 9 + 901 pp. London, E. & F. Spon, Ltd.; New York, Spon & Chamberlain, 1908. $2.00 net.

In the preface to the original edition, dated 1862, this work is stated to be a concise and comprehensive manual of data for the various and rapid calculations so constantly needed in the ordinary operations of the engineer. The preface of the present edition states that the authors have made very considerable additions and alterations, suggested by their progressive experience, by the advance of engineering science, and by professional criticism, the substitution of new matter for that which has become obsolete and the condemnation of those portions unnecessarily diffuse, enabling them to do this without increase in bulk.

PURE MILK AND THE PUBLIC HEALTH.

A Manual of Milk and Dairy Inspection. By Archibald Robinson Ward, With Two Chapters by Myer Edward Jaffa. Cloth, 9 x 6 in., illus., 13 + 218 pp. Ithaca, N. Y., Taylor & Carpenter, 1909. $2.00 net.

The author has attempted, he states in his preface, the task of assembling, for the information of the health officer and others directly concerned in the crusade for better milk, the facts essential in coping with the difficulties encountered in carrying on efficient and comprehensive measures for milk and dairy inspection. Each chapter is followed by a list of references bearing on the subjects discussed, and referred to in the text by numbers in parentheses. The Contents are: The Contamination of Milk; Changes in Milk Caused by Bacteria; Epidemic Diseases Transmitted by Milk; Bovine Tuberculosis and Other Cattle Diseases; Municipal Sanitary Control of Milk; Pasteurization of Milk; Microscopic Tests of Milk; Bacteriological Examination of Milk; Certified Milk; The Analysis of Milk; Adulteration of Milk; Figures in Text; Appendix A, Dairy Ordinance of Berkeley, Cal.; Appendix B, Chicago Tuberculin Test Ordinance; Appendix C, Duluth Milk Ordinance; Appendix D, Blank for Reporting Existence of Infectious Diseases; Appendix E, Poster for Dairy Barns; Index.

THE REINFORCED CONCRETE POCKET BOOK.

Containing Useful Tables, Rules and Illustrations for the Convenient Design, Rational Construction and Ready Computation of Cost of Reinforced Concrete Girders, Slabs, Footings, Columns, Buildings, Retaining Walls, Tanks, Grain Elevators, Coal Bins, Water Pipes, Sewers, Dams, Bridges, Smoke Stacks, Piles, etc., etc. By L. J. Mensch, M. Am. Soc. C. E. Leather, 6¼ x 4¼ in., illus., 216 pp. San Francisco, The Author, 1909. $10.00.

The object of this pocket book is to place before the public, in as concise a form as possible, practical information in regard to reinforced concrete construction, and the author also expresses the hope that the book will promote the use of reinforced concrete and be the medium of its standardization. It is not a text-book, he says, and a good knowledge of mechanics and of building construction is indispensable to its rational use. The tables, of which there are a great number, are said to have been very carefully calculated and checked repeatedly by slide-rule, and most of them, having been calculated some years ago, have also been thoroughly tested by practical application. The preface further states that the compact form of the book has compelled the author to confine the information given to those subjects which cannot readily be found in other books and to those applications of reinforced concrete which are most used.

* Unless otherwise specified, books in this list have been donated by the publisher.

PRACTICAL ALTERNATING CURRENTS AND ALTERNATING CURRENT TESTING.

By Charles F. Smith. Third Edition, Revised and Enlarged. Cloth, 8¾ x 5½ in., illus., 15 + 455 + 6 pp. Manchester, England, The Scientific Publishing Company, 6 shillings net.

This volume is an attempt, according to the preface, to introduce the student to the main principles of alternating currents and alternating-current machinery from an experimental, rather than from an abstract theoretical, standpoint. The experiments are described in great detail, and the author hopes that the book may be of use to the young engineer engaged in handling or testing alternating-current machinery, as well as to the student in the laboratory. The present edition, the preface states, has been fundamentally revised, and, in order to make room for new matter, many sections of the earlier edition have been much condensed. Many additions have been made to the chapters on transformers and alternators, while the chapter on induction has been largely rewritten, and a chapter on single-phase motors added. The Contents are: Alternating Electromotive Force and Current; Impedance; Power and Power-Factor; Virtual Value of an Alternating Current; Effect of Capacity; The Transformer; Alternators; Synchronous Motors; The Polyphase Circuit; The Rotary Converter; The Polyphase Induction Motor; Single-Phase Motors; The Composition of Waves; Index.

AZIMUTH.

By George L. Hosmer. Leather, 7½ x 4¾ in., illus., 5 + 73 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $1.00 net. (Donated by the Author and the Publishers.)

The preface of this volume states the author's purpose to be to present, in compact form, certain approximate methods of determining the true bearing of a line, with the necessary rules and tables arranged in a simple manner, so that they will be useful to the practical surveyor. This is a handbook, it is stated, rather than a textbook, and for this reason many subjects, ordinarily included in books on practical astronomy, but not essential in learning to make the observations here described, have been omitted. The author has endeavored, he declares, to present the subject so that a person unfamiliar with astronomy will be able to apply these methods and obtain satisfactory results without taking time to master completely the underlying theory. The rules and tables have been put in compact form, so that the book may be carried into the field, and, while "the methods presented are not new, much of the valuable matter written on this subject is so scattered that it is difficult to find, in one small book, all that would be needed by the surveyor in making azimuth observations, and these are the reasons given by the author for presenting this volume."

THE NEW BUILDING ESTIMATOR.

A Practical Guide to Estimating the Cost of Labor and Material in Building Construction, from Excavation to Finish, With Various Practical Examples of Work Presented in Detail, and With Labor Figured Chiefly in Hours and Quantities: A Handbook for Architects, Builders, Contractors, Appraisers, Engineers, Superintendents, and Draftsmen. By William Arthur. Cloth, 6¾ x 4½ in., illus., 4 + 424 pp. New York, David Williams Company, 1909. $2.50 net.

This book is really a fifth edition, revised and enlarged, of "The Building Estimator," the first edition of which was published in 1904. In the preparation of the material for the work, the author states that he has drawn not only upon his own wide experience as an estimator and builder, but that he has obtained from scores of others, who were best qualified to furnish information, the data on quantities and costs pertaining to a great number of representative structures, large and small. The prices in this edition are said to be brought up to date, and much new tabulated matter has been added.

EIN NEUES SCHNELLBAHNSYSTEM: VORSCHLÄGE ZUR VERBESSERUNG DES PERSONEN-VERKEHRS.

Von August Scherl. Boards, 11¼ x 14 in., illus., 6 + 122 pp. Berlin, August Scherl, 1909.

The author of this work states that the ideas and propositions here laid before the public have been developed by many years of observation, and that by them he hopes to offer a remedy for a deep-seated evil. The reader will find, he states, that the modern transit problem is taken up from an entirely new side, by entirely

new technical methods, which, in their details, have been worked out by prominent specialists, tested by the first authorities, and pronounced to be the traffic methods of the future. The Contents are: Kapitel I, Die Krisis im Gegenwärtigen Eisenbahnwesen; Kapitel II, Das Neue System—Allgemeine Prinzipien des Personenverkehrs, Die Organisation, Die Technische Durchführung des Neuen Systems (Das Einschienensystem, Das Rollende Material, Die Strecke, Die Bahnhöfe), Der Betrieb; Kapitel III, Wirtschaftliche und Soziale Ausblicke.

EXPLOITATION DES PORTS MARITIMES.

Par De Cordemoy. Leather, 7½ x 5 in., 8 + 560 pp. Paris, H. Dunod et E. Pinat, 1909. 9 francs.

The Chapter headings are: Généralités; Régime des Ports en France; Ports Français et Etrangers; Les Ports Francs; Installations Générales-Hangars; Magasins et Quais; Manutention dans les Ports; Machinerie des Ports; Divers; Réparation des Navires—Cales; Réparation des Navires—Formes de Radoub; Réparation des Navires—Docks Flottants; Fermeture des Bassins de Radoub; Des Droits de Port; Développement de Quelques Ports; Règlements.

"HÜTTE": DES INGENIEURS TASCHENBUCH.

Herausgegeben vom Akademischen Verein Hütte. Zwanzigste Auflage. Leather, 7½ x 4½ in., illus., 3 v. Berlin, Wilhelm Ernst & Sohn, 1908-09. 20 marks.

The preface, which is signed by the Chairman of the Pocketbook Committee of the "Akademischen Verein Hütte," states that the first edition of the "Hütte" was issued in one volume, divided into three parts, each forming a complete whole. Later editions were issued in two volumes, but with the present edition, the compilers have returned to the original division. The first two of these volumes, which contain material for the mechanical engineer and the shipbuilder, were issued together in 1908, and the third, devoted to matters of interest to the construction engineer, has been issued separately this year. The preface states that much valuable matter has been added, and important chapters have been entirely rewritten; that the aim constantly before the Committee was to keep in close touch with the leading practice of the day and to select without prejudice only such data, both in text and illustration, as would be useful in actual practice. The Contents are: Abteilung I, Mathematik; Mechanik Starrer Körper; Mechanik Tropfbar Flüssiger Körper; Wärme; Festigkeitslehre; Stoffkunde; Eisenhüttenkunde; Maschinenteile; Anhang. Abteilung II, Kraftmaschinen; Arbeitsmaschinen; Schiffbau und Schiffsmaschinenbau; Eisenbahnbetriebsmittel; Elektrotechnik; Abteilung III, Vermessungskunde; Hochbau; Lüftung und Heizung; Strassenbau; Wasserversorgung; Städteentwässerung; Statik der Baukonstruktionen; Eisenbetonbau; Brückenbau; Eisenbahnbau; Drahtseilbahnen; Zahnradbahnen; Wasserbau; Gasfabrikation; Fabrikanlagen; Sachverzeichnis der I., II., und III. Abteilung.

BROWN'S DIRECTORY OF AMERICAN GAS COMPANIES: GAS STATISTICS, 1909.

Compiled and Corrected Annually by E. C. Brown. Cloth, 10 x 6½ in., 417 pp. New York, Press of the *Progressive Age*, 1909. $5.00.

This directory contains statistics of gas companies in the United States, Canada, South America, and Cuba, and of electrical companies where they are operated under the same name as the gas companies. Acetylene, gasolene and natural gas companies are included, also a list of parent or operating companies. There is an appendix containing brief historical data and financial status of the companies reported, a list of officers and members of gas associations in the United States, and a bibliography of books for gas men, divided by subject. The arrangement of the book is alphabetical.

EXERCISES IN SURVEYING FOR FIELD WORK AND OFFICE WORK.

With Questions for Discussion; Intended for Use in Connection with the Author's Book, "Plane Surveying." By John Clayton Tracy. Morocco, 7½ x 5 in., 14 + 169 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $1.00 net.

Part I of this book, which, as the title indicates, is to be used with the author's book, "Plane Surveying," consists of exercises in field work grouped according to the nature of the work, while Part II consists of exercises in office computations and plotting. The whole, the author asserts, constitutes a thorough introductory course in principles and methods. Each exercise begins with a refer-

ence to "Plane Surveying," and nearly all end with series of questions, which, the author states, should prove particularly valuable to young men who are seeking to perfect themselves in the knowledge of surveying. The Contents are: Part I, Field Work—Introductory Exercises; Exercises in Chaining; Exercises in the Use of the Transit; Exercises in Running Transit Lines; Transit Surveys; Special Problems in Transit Surveying; Exercises in Leveling; Special Problems in Leveling; Exercises in the Use of the Compass; Exercises in the Use of the Stadia; Exercises in the Use of the Plane Table; Exercises in Topographic Surveying; Exercises in Determining the True Meridian; A Study of Surveying Instruments; Adjustments; Part II, Office Work—General Methods of Computation; Calculation of Bearings; Latitudes and Departures; Triangulation; Omitted Measurements; Areas; Earthwork Calculations; Exercises in Plotting; Questions Pertaining to Mapping. There is also a detachable blank Field Note Book at the end of the book, of which duplicates are obtainable.

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The Beautifying of San Jose, A Report to the Outdoor Art League. By Charles Mulford Robinson. San Jose, 1909.

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The First International Road Congress, Paris, 1908. By G. Montagu Harris and H. T. Wakelam. London, Wyman & Sons, Limited.

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Gesundheits Ingenieur: Zeitschrift für die Gesamte Städtehygiene. Jahrgänge 26–31: 1903, '05–08. München und Berlin, R. Oldenbourg, 1903–08.

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Chaudières Marines et Accessoires, avec Notes de Résistance des Matériaux et de Thermodynamique. Par L. Jauch et A. Masméjean. Two Volumes. Toulon, J. Alté; Paris, H. Dunod et E. Pinat, 1909.

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The Building Foreman's Pocket Book and Ready Reference. By H. G. Richey. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Tables of the Properties of Steam and other Vapors and Temperature-Entropy Table. By Cecil H. Peabody. Eighth Edition, Rewritten. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Practical Testing of Gas Meters. By C. H. Stone. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Railroad Promotion and Capitalization in the United States. By Frederick A. Cleveland and Fred Wilbur Powell. New York, Longmans Green and Co., 1909.

Theory and Calculation of Transient Electric Phenomena and Oscillations. By Charles Proteus Steinmetz. New York, McGraw Publishing Company, 1909.

Cours de Ponts Métalliques, Professé à l'École Nationale des Ponts et Chaussées. Par Jean Résal. Tome 1, Encyclopédie des Travaux Publics. Paris, Librairie Polytechnique, Ch. Beranger, Éditeur, 1908.

Handbuch der Ingenieurwissenschaften, Vol. 3, Part IV. Die Baumaschinen. Von F. Lincke. Zweite vermehrte Auflage. Leipzig, Wilhelm Engelmann, 1908.

Disposal of Liquid Refuse from Distilleries: Sixth Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) May Properly Be Adopted. London, Wyman and Sons, Limited, 1909.

Principles of Mining, Valuation, Organization and Administration, Copper, Gold, Lead, Silver, Tin and Zinc. By Herbert C. Hoover. New York, Hill Publishing Company, 1909.

The Westinghouse E-T Air Brake Instruction Pocket Book; A Complete Work Explaining in Detail the Improved Westinghouse Locomotive Air-Brake Equipment, Including Both the No. 5 and the Latest Perfected No. 6 Style; Contains Examination Questions and Answers. By W. W. Wood. New York, The Norman W. Henley Publishing Co., 1909.

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Heavy Electrical Engineering. By H. M. Hobart. New York, D. Van Nostrand Company, 1908.

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BAUER, JACOB LOUIS. 130 Broad St., Elizabeth, N. J.	Assoc. M.	Feb. 5, 1896
	M.	June 1, 1909
BERRY, JOHN BENNINGTON. Chf. Engr., C., R. I. & P. Ry. Co., La Salle Station, Chicago, Ill.		May 4, 1909
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BETTS, ROMEO THOMPSON. Asst. Engr., Dept. Docks and Ferries, Pier A, N. R., New York City	Assoc. M.	Oct. 3, 1900
	M.	Feb. 2, 1909
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BREWER, BERTRAM. City Engr. and Supt. of Sewers, Waltham, Mass.		June 1, 1909
BROWNE, JAMES SIMPSON. Div. Engr., N. Y., N. H. & H. R. R., 409 Union Station, Providence, R. I.	Assoc. M.	Oct. 4, 1893
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CORNELL, JOHN NELSON HAYWARD. 60 Wall St., New York City	Jun.	May 1, 1889
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	M.	May 4, 1909
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Fenn, William Henry. Room 226, DuPont Bldg., Wilmington, Del.		July 1, 1909
Ferguson, John Berton. Hagerstown, Md.		July 1, 1909
Fletcher, Austin Bradstreet. 15 Ashburton Pl., Boston, Mass		June 1, 1909
French, James Adams. Care, U. S. Reclamation Service, Engle, N. Mex.		July 1, 1909
Getman, Frank Lawton. Engr. in Chg. of Office, Cuban Eng. & Contr. Co., 438 Lonja del Comercia, Havana, Cuba.	Assoc. M. M.	April 5, 1905 May 4, 1909
Goodwin, George Estyn. Project Engr., U. S. Reclamation Service, Family, Mont.	Assoc. M. M.	Jan. 8, 1908 July 1, 1909
Grace, Arthur. Medicine Hat, Alta., Canada		July 1, 1909
Hardy, Harry. Care, Panama R. R., Colon, Panama	Assoc. M. M.	June 1, 1892 June 1, 1909
Harkness, George Edward. 60 City Hall, Boston, Mass.		June 1, 1909
Haslam, Erwin Ernest. Box 235, Tupper Lake, N. Y.		July 1, 1909
Hawgood, Harry. Cons. Engr., 722 H. W. Hellman Bldg., Los Angeles, Cal.		May 4, 1909
Hewett, William Sherman. Box 1004, Minneapolis, Minn		June 1, 1909
Hittell, John Benjamin. Chf. Engr. of Streets, Board of Local Impvts., 200 Randolph St., Room 300, Chicago, Ill.		July 1, 1909
Hoyt, John Clayton. Asst. Chf. Hydrographer, U. S. Geological Survey, 1330 F St., N. W., Washington, D. C.	Assoc. M. M.	May 4, 1904 June 1, 1909
Hyde, Charles Gilman. San. and Hydr. Engr.; Associate Prof., San Eng., Univ. of California, Berkeley, Cal.	Assoc. M. M.	April 2, 1902 June 1, 1909
Jacob, Thomas Nottingham. Chf. Engr., The East Side Levee and San. Dist., 205 Sexton Bldg., East St. Louis, Ill.		May 4, 1909
Jacobs, Joseph. Care, Porto Rico Irrig. Service, Guayama, Porto Rico.	Assoc. M. M.	Sept. 4, 1901 April 6, 1909
James, William Atlee. Box 85, Kenora, Ont., Canada		July 1, 1909
Johnson, Arthur Newhall. State Highway Engr., Springfield, Ill.	Assoc. M. M.	Sept. 2. 1903 July 1, 1909
Jones, Howard Murfree. Room 48, Cole Bldg., Nashville, Tenn		June 1, 1909
Lake, Edward Nelson. 321 Center St., Wheaton, Ill.		July 1, 1909

MEMBERS (*Continued*).		Date of Membership.
LEONARD, JOHN BUCK. 813 Sheldon Bldg., San Francisco, Cal		June 1, 1909
MAIN, CHARLES THOMAS. 45 Milk St., Room 911, Boston, Mass		July 1, 1909
MANLEY, LAURENCE BRADFORD. 22 Garden St., West Roxbury, Mass		July 1, 1909
MARTIN, EDGAR DARWIN. 172 Washington St., Chicago, Ill.		July 1, 1909
MASON, FRANCIS. Res. Engr., P. T. & T. R. R. Co., 345 East 33d St., New York City	Jun.	April 3, 1900
	Assoc. M.	April 1, 1903
	M.	July 1, 1909
MEREDITH, WYNN. 907 Union Trust Bldg., San Francisco, Cal		June 1, 1909
MITCHELL, WILLIAM SELBY. U. S. Engr. Office, Custom House, St. Louis, Mo		June 1, 1909
MONROE, RICHARD. U. S. Asst. Engr., U. S. Engr. Office, Rock Island, Ill		June 1, 1909
O'BRIEN, ARTHUR. City Engr., Utica, N. Y		July 1, 1909
PAGE, LOGAN WALLER. Director, Office of Public Roads, U. S. Dept. of Agriculture, 2223 Mass. Ave., Washington, D. C		July 1, 1909
PARKER, FREDERICK YANCY. U. S. Engr. Office, Custom House, St. Louis, Mo		June 1, 1909
PATTERSON, WILLIAM RODNEY. 1448 Monadnock Blk., Chicago, Ill		May 4, 1909
PERRY, CHAUNCY RUSCH. Diamond Pond, *via* Colebrook, N. H	Assoc. M.	Feb. 1, 1905
	M.	June 1, 1909
PEYTON, JOHN HOWE. Cons. Engr., North Coast R. R. Co., Spokane, Wash		June 1, 1909
PHILLIPS, WILLIAM RENTON. 419 Lumber Exchange Bldg., Portland, Ore		June 1, 1909
PITCHER, SAMUEL HENRY. 418 Main St., Worcester, Mass.		Mar. 2, 1909
RAIKES, HUGH PERCIVAL. 63 Temple Row, Birmingham (also 15 Victoria St., Westminster, London), England	Assoc. M.	June 6, 1906
	M.	July 1, 1909
ROHRER, GRANT. 299 Broadway, New York City		July 1, 1909
ROURKE, LOUIS KEEGAN. Asst. Div. Engr., Central Div., Isthmian Canal Comm., Empire, Canal Zone, Panama		July 1, 1909
SAYLES, EARLE WILLOUGHBY. City Engr., City Hall, Watertown, N. Y		June 1, 1909
SCHROEDER, CHRISTOFFER. 6509 LaFayette Ave., Chicago, Ill		June 1, 1909
SELLEW, FRANCIS LIBBY. Project Engr., U. S. Reclamation Service, Yuma, Ariz	Assoc. M.	Dec. 5, 1906
	M.	July 1, 1909
THOMAS, DAVID GORTON. Chf. Engr., Denver Union Water Co., Denver, Colo		June 1, 1909

MEMBERS (*Continued*).

	Date of Membership.
TORRANCE, WILLIAM MARTIN. Constr. Engr., Merrill-Ruckgaber Co., 50 Church St., New York City	July 1, 1909
TOWER, MORTON LOUDON. U. S. Engr. Office, 665 Monadnock Bldg., San Francisco, Cal.	Assoc. M. May 6, 1908 M. July 1, 1909
VAN NORDEN, RUDOLPH WARNER. Cons. Engr., 912 Union Trust Bldg., San Francisco, Cal	July 1, 1909
VINCENT, EDWIN DERICKSON. Res. Engr., U. S. Reclamation Service, Laguna Dam, *via* Yuma, Ariz	May 4, 1909
WALDRON, SAMUEL PAYSON. Care, Am. Bridge Co., 30 Church St., New York City	Assoc. M. Jan. 7, 1903 M. May 4, 1909
WILLIAMS, FRIEND PITTS. Res. Engr., N. Y. State Barge Canal, 352 Guy Park Ave., Amsterdam, N. Y	July 1, 1909
WILLOUGHBY, JULIUS EDGAR. Engr. of Constr., L. & N. R. R. Co., 9th and Broadway, Louisville, Ky	June 1, 1909
WINDSOR, PHILIP BRUNDAGE. 1 Madison Ave., New York City	July 1, 1909

ASSOCIATE MEMBERS

ACKENHEIL, ALFRED CURTIS. Box 53, Rockville Center, N. Y	April 6, 1909
AREND, ALBERT CORNELIUS. Cons. Engr., Room 691, Brandeis Bldg., Omaha, Nebr	Feb. 2, 1909
BARKER, CHARLES WHITNEY TILLINGHAST. Pres., Field, Barker & Underwood, Inc., Engrs. and Contrs., 718 Arcade Bldg., Philadelphia, Pa	Jun. June 6, 1905 Assoc. M. June 1, 1909
BEACH, WILLIAM NICHOLAS. 26 Cortlandt St., New York City	July 1, 1909
BURNS, LOUIS ANDREW. Barge Canal Office, Rome, N. Y	July 1, 1909
BURTON, WILLIAM. Res. Engr., Detroit River Tunnel Co., Windsor, Ont., Canada (Res., 108 Charlotte Ave., Detroit, Mich.)	Jun. Oct. 3, 1905 Assoc. M. May 4, 1909
CAMPBELL, DUNCAN HUGH. Care, Montana R. R. Co., Dorsey, Mont	July 1, 1909
CAREY, EDWARD GILMAN. 688 Dawson St., New York City.	July 1, 1909
CHARNLEY, WALTER. Caixa 58, Rio Grande, Rio Grande do Sul, Brazil	May 4, 1909
CLAPP, FRANK LEMUEL. Cornwall-on-Hudson, N. Y	May 4, 1909
CONWAY, JOHN SEBASTIAN. Asst. Superv. Engr. U. S. Reclamation Service, North Yakima, Wash	May 4, 1909
COVERT, CLERMONT C. Dist. Engr., U. S. Geological Survey, Room 18, Federal Bldg., Albany, N. Y	July 1, 1909
DAVIS, JOHN CHARLES. Dean of Coll. of Applied Science and Eng., Marquette Univ., Milwaukee, Wis	July 1, 1909
EDELEN, THOMAS JEFFERSON STONE. 187 Garry St., Winnipeg, Man., Canada	July 1, 1909

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
ELLIOTT, MALCOLM. Prin. Hydrographer, Office of Chf. Engr., Isthmian Canal Comm., Culebra, Canal Zone, Panama	Jun.	Nov. 5, 1907
	Assoc. M.	April 6, 1909
ETCHEVERRY, BERNARD ALFRED. Agriculture Bldg., Univ. of California, Berkeley, Cal		June 1, 1909
FOUNTAIN, THOMAS LILLY. Civ., San., Elec. and Mech. Engr. (The Fountain-Shaw Eng. Co.), Binz Bldg., Houston, Tex	Jun.	April 2, 1907
	Assoc. M.	April 6, 1909
FRYE, HARLEY EDGAR. U. S. Engr. Office, Zanesville, Ohio		June 1, 1909
GERSBACH, EDWARD CHARLES. Care, U. S. Reclamation Service, Browning, Mont		June 1, 1909
GIBBS, ELBERT ALLAN. Asst. Engr., McClintic-Marshall Constr. Co., Pittsburg, Pa	Jun.	Mar. 6, 1906
	Assoc. M.	July 1, 1909
GILBERT, ARCHIBALD MARVINE. Asst. Engr., Twin Falls Salmon River Land & Water Co., Twin Falls, Idaho		May 4, 1909
GILLEN, WALTER JOSEPH. Asst. Engr., Board of Water Supply, New York City, Brown Station, Ulster Co., N. Y.	Jun.	Oct. 31, 1905
	Assoc. M.	July 1, 1909
GRAM, LEWIS MERRITT. 1047 Spitzer Bldg., Toledo, Ohio		June 1, 1909
GRAY, HENRY LILBURN. Engr. of R. R. Comm. of Wash., Olympia, Wash		July 1, 1909
HARDESTY, JAMES ROBERT. Care, Virginia Bridge & Iron Co., Roanoke, Va		Feb. 2, 1909
HARTMAN, RUSSELL THEODORE. Care, Des Moines Bridge & Iron Works, Des Moines, Iowa		May 4, 1909
HATTON, HERBERT WATSON. Care, T. Chalkley Hatton, 302 Equitable Bldg., Wilmington, Del	Jun.	April 4, 1905
	Assoc. M.	June 1, 1909
HAZELTON, WILLIAM SYLVESTER. 145 W. Euclid Ave., Detroit, Mich		May 4, 1909
HECK, NICHOLAS HUNTER. U. S. Coast Survey, Washington, D. C		July 1, 1909
HILL, GEORGE SAMUEL. Cons. Engr. (Lathrop & Hill), 1305 Call Bldg., San Francisco, Cal	Jun.	Jan. 3, 1907
	Assoc. M.	June 1, 1909
HILTON, HARRY LEGRAND. Box 604, Cornwall-on-Hudson N. Y		June 1, 1909
HUFF, CLYDE LESLIE. Care, The Arnold Co., Moore, Idaho		July 1, 1909
HUNTER, THOMAS BENTON. Res. Engr., Monterey County Water Works, Monterey, Cal	Jun.	April 4, 1905
	Assoc. M.	July 1, 1909
JORGENSEN, LARS RASMUS. 1406 Chronicle Bldg., San Francisco, Cal		May 4, 1909
KELLEY, MATTHEW DETOBIN. Licensed Civ. Engr. and Surv., Lander, Wyo	Jun.	Sept. 3, 1907
	Assoc. M.	June 1, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
KING, CLIFFORD MARSHALL. City Engr., Sandusky, Ohio	Jun. Mar. 6, 1906 Assoc. M. July 1, 1909
KISSACK, ALFRED BROUGHTON. 903 S. Third St., Leavenworth, Kans	May 4, 1909
LAFLER, WILLIAM ARTHUR. P. O. Box 351, Whitesboro, N. Y	May 4, 1909
LAMONT, CLARENCE BOOTH. 1009 Cherry St., Seattle, Wash.	May 4, 1909
LAURGAARD, OLAF. Asst. Engr., U. S. Reclamation Service, Conconully, Wash	June 1, 1909
LINEBERGER, WALTER FRANKLIN. Apartado 333, Torreón, Coah., Mexico	May 4, 1909
LONGLEY, FRANCIS FIELDING. Care, City Engr., Toronto, Ont., Canada	Jun. Feb. 28, 1905 Assoc. M. Mar. 2, 1909
MCBEE, VARDRY ECHOLS, JR. Care, James Team, Lugoff, S. C	Oct. 7, 1908
MARTIN, BERTRAND CLIFFORD. Asst. Dist. Engr., N. Y. C. & H. R. R. R. Co., 163 Flower Ave., East, Watertown, N. Y	May 4, 1909
MATLAW, ISAAC SOLON. Asst. Engr., Dept. of State Engr. and Surv., Vischer Ferry, N. Y	Jun. Oct. 2, 1906 Assoc. M. July 1, 1909
MEIER, WILLIAM. 81 Alice Pl., Chicago, Ill	June 1, 1909
MOSS, ROBERT FAULKNER. Columbia, Mo	Jun. Nov. 1, 1904 Assoc. M. Mar. 2, 1909
NASH, FRANK DANA. Div. Engr., Bureau of Public Works, Manila, Philippine Islands	May 4, 1909
PADDOCK, HOWARD CHARLES. 1916 Eighty-fifth St., Brooklyn, N. Y	June 1, 1909
PATTERSON, CLAIR BRANDON. Res. Engr., L. S. & M. S. Ry., Darrowville, Ohio	July 1, 1909
PELLISSIER, GEORGE EDWARD. Chf. Engr., Goldschmidt Thermit Co., 90 West St., New York City	Jun. Feb. 28, 1905 Assoc. M. July 1, 1909
PETTEBONE, LAUREN AUGUSTUS. 307 Buffalo Ave., Niagara Falls, N. Y	Feb. 2, 1909
PHILLIPS, WILLIAM HALE. Box 1023, Spokane, Wash	Jun. Nov. 1, 1904 Assoc. M. May 4, 1909
POND, HARRY BRADFORD. 1432 Crotona Park, East, New York City	May 4, 1909
POOLE, JOHN HUDSON. Capt., Corps of Engrs., U. S. A., Room 150, War Dept., Washington, D. C	June 1, 1909
PRATT, JOEL MARSH. 24 South Catherine St., Mobile, Ala.	July 1, 1909
RHOADES, THEODORE ECKFORD. Chf. Engr., Whitney-Steen Co., 135 Broadway, New York City	June 1, 1909
RICE, RAY HOWARD. The Monitor, Granby St., Norfolk, Va.	July 1, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
SHELEY, HORACE WEST. Civ. and Min. Engr., P. O. Box 34, Salt Lake City, Utah.....	Jun. Sept. 5, 1905 Assoc. M. July 1, 1909
SMITH, WILLIAM ERNEST. Care, Eng. Constr. Co., Washington, Kans.....	Dec. 1, 1908
SWARTWOUT, ROY ADOLF. Chf. Engr., Western Bridge & Constr. Co., 340 Bee Bldg., Omaha, Nebr.....	Jun. May 2, 1905 Assoc. M. July 1, 1909
SWEENEY, HARRY CLINTON. 790 President St., Brooklyn, N. Y.....	July 1, 1909
THOMAS, WILLIAM EDWARD. Colonial Hall, Jamaica, N. Y.	July 1, 1909
TILT, GARRET EDWARD. Milliken, N. Y.....	June 1, 1909
TUDBURY, WARREN CHAMBERLAIN. Asst. Engr., N. Y. C. & H. R. R. R. Co., 145 Washington St., Buffalo, N. Y..	June 1, 1909
WALDRON, ALBERT EDWIN. Capt., Corps of Engrs., U. S. A., U. S. Engr. Office, P. O. Box 1092, Dallas, Tex.....	July 1, 1909
WILLARD, NORMAN RAND. Camp La Prele, Inez, Wyo.....	June 1, 1909
WILLIAMS, JACOB PAUL JONES. Asst. Engr., Designer, McClintic-Marshall Constr. Co., 10 Trenton Ave., Swissvale, Pa.....	July 1, 1909
WINTERHALTER, LEO P. Asst. Engr. on Constr., Osborn Eng. Co., Cleveland, Ohio.....	July 1, 1909
YAPPEN, ADOLPH. 430 North Francisco Ave., Chicago, Ill..	June 1, 1909

ASSOCIATE

HAYS, JOHN COFFEE. Visalia, Cal.....	May 4, 1909

JUNIORS

ASHBROOK, CHESTER DANIEL. Care, King County Engr.'s Office, Seattle, Wash.....	May 4, 1909
BERGMAN, HARRY MONTIFIORE. 2422 Morris Ave., Bronx, New York City.....	May 4, 1909
BESWICK, JAMES EVERETT. Cold Spring, Putnam Co., N. Y.	July 1, 1909
BISHOP, LYMAN EDGAR. 3787 Franklin, Denver, Colo.....	May 4, 1909
BROOKS, JOSIAH RICHARDSON. 603 N. Main St., Mt. Vernon, Ohio.....	Feb. 2, 1909
BURR, MYRON CARLOS. (Burr & Ferguson), 216 Loo Bldg., Vancouver, B. C., Canada.....	May 4, 1909
CHAPMAN, ARNOLD GOODWIN. 52 Chestnut St., Albany, N. Y.....	July 1, 1909
CUTLER, STANLEY GARDNER. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands.....	May 4, 1909
HOLLAND, HOWARD KINGSBURY. 207 New Eng. Bldg., Ann Arbor, Mich.....	June 1, 1909
HORRIGAN, WILLIAM JAMES. 405 Ford Bldg., Wilmington, Del.....	June 1, 1909
HOWE, FRANK RAY. 66 Jamaica Ave., Flushing, N. Y....	June 1, 1909

JUNIORS (*Continued*).	Date of Membership.
HOWES, FRANKLIN JOHNSON. 34 Clinton Ave., Rochester, N. Y.	June 1, 1909
JONES, BENJAMIN EARL. Care, Office of Public Roads, Washington, D. C.	April 6, 1909
KRIEGER, ALBERT AUGUST. 1141 Cherokee Rd., Louisville, Ky	June 1, 1909
LARIMER, ROBERT SHERMAN. Estes Park, Colo.	June 1, 1909
LEMCKE, KARL WOLFGANG. Box 383, Ambridge, Pa.	June 1, 1909
MYERS, GEORGE HIGGINS. 219 McPhee Bldg., Denver, Colo.	June 1, 1909
NICHOLS, JOHN ROBERT. Hotel Buckminster, Boston, Mass.	June 1, 1909
NORDWELL, ALFRED WORCESTER. 20 Shepard St., Cambridge, Mass	June 1, 1909
RANDOLPH, ROBERT ISHAM. 164 Dearborn St., Room 748, Chicago, Ill.	July 1, 1909
ROLLINS, ANDREW PEACH. Eagle Lake, Tex.	May 4, 1909
SIMS, HARVEY HILLYER. 154 Gordon St., Atlanta, Ga.	May 4, 1909
SMALL, GILBERT. 428 Lexington St., Waltham, Mass.	June 1, 1909
SMITH, JOSEPH. Care, Board of Water Supply, Brown Station, N. Y.	Feb. 2, 1909
SUAREZ Y CORDOVES, PATRICIO ANDRES. Gen. Mgr., Compañia Eléctrica de Alumbrado y Traccion, Apartado 110, Santiago de Cuba, Cuba.	April 6, 1909
TEICHERT, ADOLPH, JR. Gen. Contr., 2401 J St., Sacramento, Cal.	July 1, 1909
TILLSON, EDWIN DELEVAN. 179 Marcy Ave., Brooklyn, N. Y.	Mar. 2, 1909
WALTON, HARRY COLLINS. 1103 Park Row Bldg., New York City.	Jan. 5, 1909
WILLIAMS, CLEMENT CLARENCE. 1133 Pleasant St., Boulder, Colo	June 1, 1909
WINTERMUTE, FERD. CLARK. 414 Second National Bank, Wilkes-Barre, Pa.	July 1, 1909
WRIGHT, THOMAS JUDSON, JR. Churchland, Va.	June 1, 1909

CHANGES OF ADDRESS

MEMBERS

ALLEN, KENNETH. Engr., Met. Sewerage Comm. of New York, 17 Battery Pl., New York City.

ALLEN, WALTER HINDS. Civ. Engr., U. S. N., U. S. Naval Magazine, Hingham, Mass.

ARGOLLO, MIGUEL DE TEIVE E. Pres., Bahia General Transportation Co., Caixa Postal 89, Bahia, Brazil.

ARMSTRONG, WALTER ROOT. Asst. Supt., Idaho Div., Ore. Short Line R. R., Nampa, Idaho.

MEMBERS (*Continued*).

ASHMEAD, FRANK MILLIGAN. Asst. to Prin. Asst. Engr., B. & A. V. Div., P. R. R., 428 Richmond Ave., Buffalo, N. Y.

BACHERT, AUGUSTUS ELLSWORTH. Gen Supt., Rockhill Iron & Coal Co., Chf. Engr., East Broad Top R. R. Co., Tyrone, Blair Co., Pa.

BACOT, WILLIAM SINCLAIR. 21 Summit Pl., Utica, N. Y.

BAKENHUS, REUBEN EDWIN. Civ. Engr., U. S. N., U. S. Naval Station, Olongapo, Philippine Islands.

BALDWIN, ERNEST HOWARD. Const. Engr., U. S. Reclamation Service, Ellensburg, Wash.

BINKLEY, GEORGE HOLLAND. Nyssa, Ore.

BISSELL, HEZEKIAH. Cons. Engr., B. & M. R. R., Boston, Mass.

BLAUVELT, LOUIS DAVID. Gen. Mgr. and Chf. Engr., The San Luis Southern Ry., Blanca, Colo.

BONZANO, MAXIMILIAN FERDINAND. Fort Totten, N. Y.

BOYDEN, HARRY CHESTER. Asst. Engr., A., T. & S. F. Ry., Del Mar, Cal.

BREWSTER, HENRY BAUM. Cons. Engr., 517 S. A. & K. Bldg., Syracuse, N. Y.

BYAM, LEROY HENRY. Asst. Engr., N. Y. C. & H. R. R. R. Co., 335 Madison Ave., New York City (Res., Edgecliff Terrace, Park Hill, Yonkers, N. Y.).

CHAMBERS, RALPH HAMILTON. (Chambers & Hone), 1 Liberty St., New York City.

CLARKE, ERNEST WILDER. Asst. Engr., Board of Water Supply, City of New York, Pleasantville, N. Y.

COLE, HOWARD JUDSON. 11 High St., Morristown, N. J.

CONDRON, THEODORE LINCOLN. Cons. Bridge and Structural Engr., 1214 The Monadnock, Chicago, Ill.

CRANDALL, CHARLES LEO. Sandwich, Mass.

CROCKER, HERBERT SAMUEL. (Crocker & Ketchum, Cons. Engrs.), 811 Seventeenth St., Suite 438, Denver, Colo.

CROSWELL, THOMAS HENRY. Chf. Engr., Spokane & Seattle Ry. Co., Astoria & Columbia River R. R. Co., East 8 Hill Ave., Spokane, Wash.

DAKIN, ALBERT HARLOW, JR. 370 St. Nicholas Ave., New York City.

DARRACH, CHARLES GOBRECHT. Cons. Engr., Anacortes, Wash.

DAUCHY, WALTER EDWARD. Railroad Contr. (Johnson, Dauchy & Carey Co.), Mandan, N. Dak.

DAWLEY, WILLIAM SANBORN. Chf. Engr., Yunnan-Szechuan & Tengyueh Ry. Co., Yunnanfu, Yunnan Prov., China.

DIEBITSCH, EMIL. Vice-Pres., The John Peirce Co., 277 Broadway, New York City (Res., 38 Burnett Pl., Nutley, N. J.).

DOSE, HENRY FREDERICK. Belleville, Ill.

ENGLE, ROBERT L. 1530 Lincoln Ave., Cincinnati, Ohio.

ERLANDSEN, OSCAR. 359 Fulton St., Jamaica, N. Y.

EWING, WILLIAM WALLACE. 10 Bridge St., New York City.

FERGUSON, JOHN NEIL. Div. Engr., Charles River Basin Comm., Lock-Gate House, cor. Charles and Leverett Sts., Boston, Mass.

FISH, JOHN CHARLES LOUNSBURY. Box 233, Stanford University, Cal.

MEMBERS (*Continued*).

FITCH, CHARLES HALL. 3031 N St., N. W., Washington, D. C.

FORGIE, JAMES. Chf. Asst. Engr., Pennsylvania R. R. Tunnels (N. R. Div.), Pennsylvania Station, 32d St. and Seventh Ave., New York City.

FULLER, WILLIAM BARNARD. Cons. Civ. Engr., 150 Nassau St., Room 1135, New York City.

GALLOWAY, JOHN DEBO. (Galloway & Markwart), First National Bank Bldg., San Francisco, Cal.

GIFFORD, ROBERT LADD. Pres., Illinois Eng. Co., Monadnock Blk. (Res., 4908 Kimbark Ave.), Chicago, Ill.

GOODWIN, JAMES BOWMAN. 400 Eagle St., Medina, N. Y.

GORDON, CHARLES EDWARD. Care, Dept. of Public Works, Manila, Philippine Islands.

GREEN, HUBERT EDWARD. Const. Engr., 633 Central Bldg., Los Angeles, Cal.

GUPPY, BENJAMIN WILDER. Bridge Engr., B. & M. R. R., Boston, Mass.

HALL, HENRY ARTHUR. 4306 North 26th St., Tacoma, Wash.

HALL, JOHN LINCOLN. Care, Purdy & Henderson, 609 Crary Bldg., Seattle, Wash.

HALL, JULIEN ASTIN. Cons. Engr., P. O. Box 191, Danville, Va.

HARTIGAN, FREDERICK LAWRENCE. Care, Edison Portland Cement Co., St. James Bldg., New York City.

HENNY, DAVID CHRISTIAN. Cons. Engr., U. S. Reclamation Service, Beck Bldg., Portland, Ore.

HEWINS, GEORGE SANFORD. Care, J. G. White & Co., North Fork, Madera Co., Cal.

HONE, FREDERIC DE PEYSTER. (Chambers & Hone), 1 Liberty St., New York City.

HOUSTON, GAVIN NELSON. Cons. and Superv. Engr., 518 Commonwealth Bldg., Denver, Colo.

HOWE, GEORGE EDWARD. Metropolitan Water Board, Boston, Mass.

HOWELL, GEORGE PIERCE. Maj., Corps of Engrs., U. S. A., Office of Chf. of Engrs., U. S. A., Washington, D. C.

JAYCOX, THOMAS WILLIAM. Cons. Civ. Engr., Room 304, Majestic Bldg., Denver, Colo.

JOHNSON, LUCIEN SAMUEL. U. S. Engr. Office, Lock No. 13, Willow, Ky.

KETCHUM, MILO SMITH. (Crocker & Ketchum, Cons. Engrs.), 811 Seventeenth St., Suite 438, Denver, Colo.

KINDER, CLAUDE WILLIAM. Brailsford, Avondale Rd., Fleet, Hants, England.

KINSLEY, THOMAS PEARSON. 600 West 138th St., New York City.

LANGTHORN, JACOB STINMAN. Div. Engr., Reservoir Dept., Board of Water Supply, City of New York, Brown Station, N. Y.

LARSSON, CARL GUSTAF EMIL. Asst. Chf. Engr., Am. Bridge Co. of N. Y., 9 Rockview Terrace, Plainfield, N. J.

LAVIS, FRED. 134 Elm Ave., Mt. Vernon, N. Y.

LION, LÉON ELIE. Asst. U. S. Engr., U. S. Engr. Office, Custom House, Norfolk, Va.

MCCARTHY, GEORGE ARNOLD. Care, Smith, Kerry & Chace, 68 McDougall Blk., Calgary, Alta., Canada.

MEMBERS (*Continued*).

McCausland, Elmer James. Prof. of Civ. Eng., Univ. of Washington, 5264 Nineteenth Ave., N. E., Seattle, Wash.

McCoy, Samuel Alexander. Care, Caughren, Winters, Smith & Co., Mandan, N. Dak.

McGrew, Anson Burlingame. U. S. Asst. Engr., Room 2106, Farmers Bank Bldg., Pittsburg, Pa.

McMinn, Thomas James (*Assistant Secretary*), 220 West 57th St., New York City (Res., 622 Ave. K, Flatbush, Brooklyn, N. Y.).

Maxim, *Sir* Hiram Stevens. Ryecotes, Dulwich Common, London, S. E., England.

Mercer, Charles Hatton. Chf. Engr., Bridge and Constr. Dept., The Pennsylania Steel Co., Steelton, Pa.

Miller, Frank. 220 Broadway, New York City.

Moberly, Frank. Barrie, Ont., Canada.

Moncrieff, John Mitchell. Pearl Bldgs., Northumberland St., Newcastle-upon-Tyne, England.

Morse, George Frederick. Asst. Engr., C. R. R. of N. J., 143 Liberty St., New York City (Res., 601 Ave. E, Bayonne, N. J.).

Mosscrop, Alfred Mitton. Rochester, N. Y.

Murphy, Daniel William. Project Engr., U. S. Reclamation Service, Washington, D. C.

Neely, William Ridley. Care, Board of Water Supply, New Paltz, N. Y.

Nicholson, Frank Lee. Chf. Engr., in Chg. of Constr. and M. of W. and Structures, Norfolk & Southern Ry. Co., Norfolk, Va.

Parker, Maurice Stiles. Cons. Engr., 328 Hutton Bldg., Spokane, Wash.

Pettigrew, Thomas. Johnstone Bldg., New York City.

Prindle, Franklin Cogswell. Civ. Engr., U. S. N.; Rear-Admiral (*Retired*), 1752 Park Rd., Washington, D. C.

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RESIGNATION

ASSOCIATE MEMBER	Date of Resignation.
HAMLIN, WILLIAM ELIOT	June 1, 1909

DEATHS

BALDWIN, HENRY FURLONG. Elected Associate Member, December 7th, 1892; Member, February 6th, 1895; died June 17th, 1909.

Buck, Lefferts Lefferts. Elected Member, February 3d, 1875; died July 17th, 1909.

Davis, Frank Leslie. Elected Member, September 6th, 1905; died June 9th, 1909.

Dimmick, John Bagley. Elected Member, July 10th, 1907; died June, 1909.

Dusenberry, Walter Lorton. Elected Junior, November 5th, 1890; died May 13th, 1909.

French, George Harrison. Elected Associate Member, December 5th, 1906; died July 9th, 1909.

Gatchell, George Samuel. Elected Member, May 7th, 1884; died June 22d, 1909.

Kernot, William Charles. Elected Member, March 6th, 1889; died March 14th, 1909.

Steele, Henry Maynadier. Elected Associate Member, April 5th, 1893; Member, December 6th, 1899; died July 5th, 1909.

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(May 11th to July 31st, 1909)

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In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England, 25c.
(22) *Iron and Coal Trades Review,* London, England, 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Pontes et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
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(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
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(71) *Journal*, Iron and Steel Inst., London, England.
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(74) *Transactions*, Inst. of Min. and Metal., London, England.
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(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
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The Constitution of Copper-Iron and Copper-Lead-Iron Mattes.* Charles H. Fulton and Ivan E. Goodner. (56) Vol. 39.
The Behavior of Calcium Sulphate at Elevated Temperatures with Some Fluxes.* H. O. Hofman and W. Mostowitsch. (56) Vol. 39.
Gayley's Invention of the Dry Blast. R. W. Raymond. (56) Vol. 39.
Experience with the Gayley Dry Blast at the Warwick Furnaces, Pottstown, Pa.* Edward B. Cook. (56) Vol. 39.
The Pearce Gold-Separation Process. Harold V. Pearce. (56) Vol. 39.
The Magnetic Separation of Ores and Its Application to Spathic Iron (Siderite) in Siegerland.* W. J. Bartsch. (Abstract of paper read before the Assoc. Berggeist.) (22) May 7.
Stock Distribution for Blast Furnaces.* J. E. Johnson, Jr. (20) May 13.
The Heroult Electric Steel Furnace.* Robert Turnbull. (Paper read before the Amer. Electrochemical Soc.) (20) May 13.
On Charcoal Blast Furnace Practice in the Ural.* M. A. Pavloff. (12) May 14.
Improvements in the Treatment of Slime by the Vacuum Filter Process.* Arthur W. Allen. (16) May 15.
The Electric Furnace Reduction of Iron Ore. Joseph W. Richards. (Paper read before the Amer. Electrochemical Soc.) (13) May 20.
The Pierce Amalgamator.* Justin H. Haynes. (45) June.
The Ohio Concentrator; a Description of the Mill at Bingham, Utah.* Leroy A. Palmer. (45) June.
Researches on the Magnetic Properties of a Series of Carbon Tungsten Steels.* Thomas Swinden. (77) June.
A Contribution to the Study of Electric Furnaces as Applied to the Manufacture of Iron and Steel.* Ch. A. Keller. (Abstract of papers read before the Faraday Soc. and the Amer. Electrochemical Soc.) (22) June 18; (73) July 2; (20) June 3.
Automatically Circulating Furnaces of the Gin Type for the Electrical Production of Steel.* Gustave Gin. (Abstract of paper read before the Faraday Soc.) (22) June 18.
The Kilker Matte Tapping Car.* F. T. Havard. (16) June 26.
The Tintic Smelter.* Leroy A. Palmer. (45) July.
Pittsburg Silver Peak Mill.* Henry Hanson. (45) July.
The Gayley Dry Air Blast. (3) July.
Methods of Refining Steel in the Electric Furnace.* J. B. C. Kershaw. (10) July.
Designing Blast Furnaces.* M. Pavloff. (22) July 9.
An All Slime Cyanide Plant at Guanajuato, Mexico. Edwin Shapley. (16) July 10.
Tailings Disposal Plant at the Wolverine Mill.* C. Kemble Baldwin. (16) July 10.
Shortening the Roasting Period for Mercury Ores.* William B. Dennis. (16) July 17.
Determining Carbon in Steel.* Warren I. Keeler. (20) July 22.
Recherches Expérimentales sur la Nature des Mattes de Cuivre.* A. Baykoff et N. Troutneff. (93) May.
Quelques Expériences sur la Trempe du Plomb et les Changements Corrélatifs de sa Duretée.* Loutchinnsky. (93) May.
Les Alliages de Cuivre Spéciaux.* Jean Escard. (33) Serial beginning May 22.

Military.

Field Guns for Destroying Dirigibles.* (19) May 29.
The Engineering of Ordnance.* A. Trevor Dawson. (Paper read before the Jun. Inst. of Engrs.) (11) Serial beginning July 2.

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Mining.

The Mining and Milling of Silver-, Lead- and Zinc-Ores at Pierrefitte Mines, France. William Waters Van Ness. (56) Vol. 39.
Sulphur Dioxide as an Agent in Fighting Mine-Fires. Walter O. Snelling. (56) Vol. 39.
Calculation of Mine Values. R. B. Brinsmade. (56) Vol. 39.
Effect of Humidity on Mine Explosions. Carl Scholz. (56) Vol. 39.
Requirements of a Breathing-Apparatus for Use in Mines.* Walter E. Mingramm. (56) Vol. 39.
The Ilsede Hütte Iron-Mines at Peine, Germany.* Lucius W. Mayer. (56) Vol. 39.
Machine Mining under Difficulties.* John Gibson. (59) Vol. 31, Pt. 5.
Dredging at Butte.* A. Floyd Bushnell. (16) May 15.
New Method of Extracting Oil from Boreholes.* Frederick A. Talbott. (16) May 15.
A Comparison of Coal-Cutting Machines. Sydney F. Walker. (16) May 22.
Electricity in Coal Mines. Robert Nelson. (Paper read before the Inst. of Min. Engrs.) (22) May 28; (57) June 4.
The Use of Concrete for Mine Support. W. R. Crane. (Paper read before the Inst. of Min. Engrs.) (22) May 28.
The Coal Dust Question in Great Britain.* Henry Hall. (16) May 29.
Suction Gas and Its Application to Mining.* George D. Stephen. (16) May 29.
New River Coal Field, W. Va.* H. H. Stoek. (45) June.
The Steel Oil Derrick.* R. B. Woodworth. (58) June.
A Model Coal Mine in Westphalia.* William S. Hall. (16) June 5.
The Panel System as Applied to Metal Mining.* H. E. West. (16) June 12.
A Novel Coal and Stone-Cutting Process.* Alfred Gradenwitz. (16) June 19.
Automatic Dumping Skip for Vertical Shafts.* Geo. C. McFarlane. (16) June 26.
Ray Consolidated Mines.* R. L. Herrick. (45) July.
Determination and Regulation of the Percentage of Gas in Mine Air-Currents.* John Gordon Smyth. (Paper read before the West Va. Coal Min. Inst.) (45) July; (16) July 3.
Some Simple Causes of Mine Explosions.* L. D. Tracy, Assoc. M. Am. Soc. C. E. (13) July 1.
Survey to Fix Position of the Water Haigh Pits. G. R. Thompson. (Paper read before the National Assoc. of Colliery Mgrs.) (22) July 2.
The Experimenting Station at Lievin, France.* Tony Callot. (16) July 3.
An Electrically Operated Hoisting Plant.* A. Gradenwitz. (16) July 10.
Electric Hoisting Equipment at Winona, Mich.* R. R. Seeber. (16) July 17.
Pumping Plant at the Tombstone Consolidated.* Elton W. Walker. (16) July 24.
Modern Gold Placer Dredging.* George B. Massey, 2d. (10) Aug.
La Lutte contre les Poussières de Houille dans les Mines.* J. Daniel. (Abstract from *Annales des Mines.*) (33) July 17.

Miscellaneous.

Notes on Canadian Forestry. Stanislas Gagné. (5) Vol. 22, Pt. 2.
Chemistry and the Conservation of our Forests and Minerals. Marston Taylor Bogert. (6) Apr.
The Salette Landslide of 1908 and some Earlier Quebec Landslides.* (13) May 27.
Temperature Determination and Control for High-Speed Tool Treatment (Pyrometers).* O. M. Becker. (9) June.
Going Value and Methods for its Computation.* John W. Alvord. (Abstract of paper read before the Amer. W. W. Assoc.) (14) June 19.
Difficult Problems which Public Utility Commissions are Endeavoring to Solve. H. C. Abell. (From *The Armour Engineer.*) (24) Serial beginning June 21.
Cost Estimating; a Discussion of Principles with Actual Estimates for Contract Work. Jean Bart Balcomb. (86) July 14.
La Radioactivité en Géologie et dans l'Atmosphère. P. Besson. (32) Mar.
Etude des Vibrations du Sol lors des Tremblements de Terre Conséquences au Point de Vue Théorique.* J. Bergeron. (32) Mar.
L'Or à Madagascar. Ch. Marquet. (32) Apr.
Les Anciennes Charrues de France.* Henri Chevalier. (32) May.

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The Rotherhithe Tunnel (Street traffic).* Edward Henry Tabor. (63) Vol. 175.
The First International Road Congress at Paris in October, 1908; what Led up to it; its Deliberations; and its Results. Austin B. Fletcher. (Paper read before the Bost. Soc. Civ. Engrs.) (1) Apr.
Road Machinery.* A. Rankin. (Paper read before the Ont. Good Roads Assoc.) (96) May 21.
Concrete Roadways. Ernest McCullough. (96) May 28.
Concrete Roadways.* Charles W. Ross. (96) June 4.
Vitrified Brick Roadways.* Will P. Blair. (From *Good Roads Magazine.*) (76) July.

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An "Accelerated Test" of Road Wear by Automobile Traffic.* (13) July 8.
The Public Lighting of the City of London; Report of the Deputation to Continental Cities. (66) July 20.
Road Materials and Some Simple Rules for Testing Them. Austin B. Fletcher, M. Am. Soc. C. E. (Paper read before the 1st Cong. of Road Builders.) (14) July 31.
Les Plantations Routières. Edmond Hermans. (30) June.
Die Bisherigen Ergebnisse der Teerungen auf den Rheinischen Provinzialstrassen bis zum Frühjahr, 1908. Görz. (Paper read before the Architekten- und Ingenieur-Verein zu Cassel.) (51) May 8.

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On the Serviceable Life and Average Annual Cost of Locomotives in Great Britain. Richard Price-Williams, M. Inst. C. E. (63) Vol. 175.
Glasgow Central Station Extension.* Donald Alexander Matheson, M. Inst. C. E. (63) Vol. 175.
The Rotherhithe Tunnel (Street traffic).* Edward Henry Tabor. (63) Vol. 175.
Straight Air Brake Equipment.* J. B. Parham. (5) Vol. 22, Pt. 2.
Improvements at the Works of the Canadian Locomotive Company, Kingston, Ont. Henry Goldmark. (5) Vol. 22, Pt. 2.
A Calibrated Speed Recorder (Train movement).* A. B. Stitzer. (2) Jan.
Influence of Ash on Value of Coal in Locomotive Service.* (61) Apr.
The Resistance of Railway Trains. Albert Frank. (88) May.
Uniformity in the Symbols used to Represent Different Types of Locomotives. P. Labryn. (From *De Ingenieur.*) (88) May.
Railroad Terminal Improvements at Providence, R. I.* George B. Francis. (Paper read before the Boston Soc. of Civ. Engrs.) (1) May.
The East Side Tunnel and its Approaches, Providence, R. I.* Edwin P. Dawley. (Paper read before the Boston Soc. of Civ. Engrs.) (1) May.
Development in Air Brakes for Railroads with a Brief Review of Past and Present Operating Conditions. W. V. Turner and S. W. Dudley. (65) May.
New 4-4-0 Engines, North-Eastern Railway.* (15) May 14.
Shop Notes from the C., St. P., M. & O. (15) May 14.
Sudbury-Kleinburg Branch of the Canadian Pacific.* (15) May 14.
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Conversion of New Canaan Branch from 500-Volt D. C. to 11 000-Volt A. C. Operation.* (17) May 15.
Electric Train Staff on the Southern Pacific.* (15) May 21.
A Suggestion for Tie Rods.* H. Herden. (15) May 21.
Railway Development in China. Arthur John Barry. (29) May 21.
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Comparative Rail Specifications. (15) May 21.
New York Passenger Terminal, Central R. R. of New Jersey.* (14) May 22.
Steel Catenary Bridges for Lines in Central New York.* (17) May 22.
The Sigafoos Tunnel Machine.* Rowland Ashford Phillip. (46) May 22.
Methods of Snow-Fighting on Railroads. A. W. Wheatley. (Paper read before the Canadian Ry. Club.) (86) May 26.
Hydraulic Construction of Large Embankments on the Chicago, Milwaukee & Puget Sound Ry.* (13) May 27.
The Jacobs-Shupert Locomotive Firebox.* (15) May 28.
The Passenger Rate of the American Railway. William S. Bronson. (15) Serial beginning May 22.
Mikado Type Locomotives Built by C., M. & St. P. Ry.* (18) May 29.
The Waterbury Improvements of the New Haven Railroad.* (14) May 29.
Chicago Track Elevation.* M. K. Trumbull. (15) June 4; (14) May 29; (13) June 3.
Mallet Articulated Compound Locomotives 2-6-6-0 Type, Virginian Railway.* (25) June; (15) June 18; (18) June 12.
Baker-Pilliod Valve Gear.* (21) June.
The Manufacture of Points and Crossings.* (21) Serial beginning June.
Zara's Bogie; Italian State Railways.* (21) June.
The Question of the Proposed Trials of Goods-Train Brakes. A. Huberti and J. Doyen. (88) June.
The Resistances and the Application of Double-Wire Transmissions for Operating Switches and Signals at a Distance.* L. H. N. Dufour. (88) June.
Low, Moderately and Highly Superheated Steam (Locomotives).* F. J. Cole. (25) June.
Mikado Type Freight Locomotive, Virginian Railway.* (25) June.
Location of Side Bearings on Tender Trucks.* Theodore H. Curtis. (25) June.
Self Clearing Ash Pans (Locomotives).* (25) June.
Flange Wear of Driving Tires (Locomotives).* R. H. Rogers. (25) June.

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Comparative Tests of Water Tube and Standard Fireboxes.* S. S. Riegel. (25) June.
Cylinders for 4-Coupled Compound 2-6-4 Engines; Austrian State Railways.* (21) June.
Mechanical Appliances for Track-Laying.* J. F. Springer. (10) June.
Some Remarkable Locomotives of 1908* J. F. Gairns. (10) June.
Details of Articulated Mallet Compound Locomotive for the Southern Pacific.* (15) June 4; (94) July.
Dimensions of New Locomotives. (96) June 4.
The Construction of the Transandine Railway.* (14) June 5.
Ten-Wheel Tank Locomotive (3-Ft. 6-In. Gauge); Dutch Indian Railway.* (11) June 11.
The Electrification of the L. B. & S. C. Railway's South London Line.* (73) June 11.
The Semaphore; Upper Left vs. Upper Right.* C. H. Morrison. (Abstract of paper read before the Ry. Signal Assoc.) (15) June 11; (18) June 26.
Simmen Automatic Railway Signal Device.* (96) June 11.
The Campbell-Olden Steel Side-Dump Car.* (18) June 19.
Locomotive Road Tests of Briquets. C. T. Malcolmson. (18) Serial beginning June 12.
Overhead Construction for the Electrification of Main Trunk Lines.* Francis S. Denneen. (Extracts from *Purdue Eng. Review.*) (8) June 12.
The Signal and Dispatching System on the Mimico Division of the Toronto & York Radial Railway.* (17) June 12.
A Locomotive Firebox without Staybolts in the Side Sheets and Crown Sheet (Atchison, Topeka & Santa Fe Ry.).* (13) June 17.
All-Electric Interlocking at Allentown.* (15) June 18.
Mechanical Stokers. (Report to the Amer. Ry. Master Mechanics Assoc.) (18) June 19.
New Locomotives for the St. Louis Southwestern Ry.* (18) June 19.
Detroit River Tunnel Locomotive.* (17) June 19; (13) June 24; (18) June 19.
New York Central Electric Locomotive.* (15) June 25.
Automatic Junction Signals at Calvin, Oklahoma.* (15) June 25.
Remarkable Plant of the St. Clair Tunnel.* Osborn Monnett. (64) June 29.
Pacific and Consolidation Type Locomotives, Chicago and Alton Railway.* (25) July; (18) June 5.
A Tabular Comparison of Notable Examples of Recent Locomotives, Arranged in Order of Total Weights. (25) July.
Locomotive Performance under Saturated and Superheated Steam.* W. F. M. Goss. (Abstract of paper read before the Master Mechanics' Assoc.) (25) July; (18) June 26.
On the Question of Improvements in Locomotive Boilers. Nadal. (88) July.
On the Question of Strengthening the Track and the Bridges with a View to Increasing the Speed of Trains. J. W. Jacomb-Hood. (88) July.
The "T. A. C." System of Electric Train Lighting.* (21) July.
Zara's Compensating Arrangement for Adjoining Axles.* (21) July.
On the Working of Italian Locomotives in Actual Service.* (21) July.
Some Comparisons of Wood and Steel Ties. J. M. Meade. (Paper read before the Roadmasters' Convention.) (87) July.
The Widening of Wemyss Bay Railway.* (11) Serial beginning July 2.
Consolidation Locomotives for the Pennsylvania.* (15) July 2.
The Loetschberg Tunnel.* H. Prime Kieffer. (15) July 2.
Grade Separation. F. L. Somerville. (96) Serial beginning July 2.
The Laying out of the Track in the Muskoka Divisional Yard.* C. D. Norton. (96) July 2.
Latest Developments in Electric Railway Apparatus. C. D. Eveleth. (Abstract of paper read before the St. Ry. Assoc. of N. Y.) (17) July 3.
Latest Improvements in Electric Railway Apparatus. J. L. Davis. (Abstract of paper read before the St. Ry. Assoc. of N. Y.) (17) July 3.
Construction Work on the C., M. & St. P. Coast Extension.* (18) Serial beginning July 3.
Splicing Sills.* (Abstract of report to Master Car Builders' Assoc.) (18) July 3.
Electrical Features of the New Northwestern Railroad Terminal in Chicago.* (27) July 8.
Complete Solution of Transmission Line with Distributed Capacity and Leakage.* Harold Pender. (27) July 8.
Special Motor-Controlling Devices in the Rolling Mill of the Indiana Steel Company, Gary, Ind.* (27) July 8.
Automatic Signal Records on the Baltimore & Ohio. (15) July 9.
Axle-Driven Head-End Electric Train Lighting System.* (15) July 9.
Compressed Gas and Electric Car Lighting.* Geo. L. Fowler. (15) July 9.
The Pressure on Locomotive Brake Blocks.* (12) July 9.

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Car Wheels.* (Report of a Comm. to the Master Car Builders Assoc.) (18) July 10; (13) July 29; (25) Aug.
Reinforced Concrete Culverts of the Schenectady Railway.* (17) July 10.
A Railway Transfer Table without a Pit.* Henry V. Miller. (13) July 15.
Wheel Foundry and Foundry Methods; Norfolk & Western.* George L. Fowler. (15) Serial beginning July 16.
The Effect of the Physical Characteristics of a Railway upon the Operation of Trains.* (15) Serial beginning July 16.
Opening of the Terminal Station of the Hudson & Manhattan Railroad.* (17) July 17.
The Earliest Balanced Locomotives.* Herbert T. Walker. (15) Serial beginning July 23.
Newburgh and North Fife Railway.* (11) July 23.
Equated Train Tonnage. J. M. Daly. (Abstract from *Ill. Central Employees Mag.*) (18) July 24.
Mallet Compound Locomotive for the Little River R. R.* (18) July 24.
Use of Alternating Currents in Railway Signaling. W. K. Howe. (Paper read before the Ry. Signal Assoc.) (15) Serial beginning July 9; (18) July 24.
The Operation of the Simplon Tunnel Railway.* (13) July 29.
Interlocking and Signalling.* E. W. Oliver. (96) July 30.
The Christiania-Bergen Railway.* (15) July 30.
Specifications for 90-Lb. Bessemer and Open Hearth Steel Rails for the Harriman Lines. (15) July 30.
Waterbury Extensions of the Connecticut Company.* (17) July 31.
Bank Versus Level Firing (Locomotives).* E. D. Nelson. (Paper read before the Am. Ry. Master Mechanics Assoc.) (25) Aug; (18) July 3.
Note sur un Procédé d'Etude des Formules d'Exploitation pour les Chemins de Fer d'Intérêt Local.* Tourtay. (43) May.
Système d'Essieux Coudés à Flasques Evidées (Locomotives).* Ch. Frémont. (92) May.
Note sur le Raccordement des Courbes et des Alignements dans les Voies de Chemins de Fer.* Collot. (38) May.
Chasse-Neige Rotatif à Vapeur de la Compagnie d'Orléans.* Conte. (38) May.
Les Ateliers Cail à Denain (Construction of Locomotives).* Louis Le Chatelier. (93) May.
Etude Complémentaire sur la Stabilité du Matériel des Chemins de Fer (Théorie des Déraillements-Profil des Bandages).* Georges Marié. (32) May.
Automotrices et Locomotives à Vapeur Surchauffée des Chemins de Fer de l'Etat wurtembergeois.* (33) May 29.
Note sur la Construction des Locomotives aux Etats-Unis, en Allemagne et en France. Louis Le Chatelier. (38) June.
Procédé de Réparation de Plaques Tubulaires au Moyen d'Agrafes et de Bagues Filetées.* F. Level et P. Engrand. (38) June.
Note sur les Chemins de Fer de l'Indo-Chine.* R. Godfernaux. (38) June.
Locomotives Pacific Compound à Quatre Cylindres de la Compagnie d'Orléans.* (33) June 19.
Note sur les Machines à Vapeur Surchauffée à Dix Roues Accouplées de la Compagnie des Chemins de Fer du Midi.* Bachellery. (38) July.
Les Nouvelles Locomotives Electriques du Tunnel du Simplon.* (33) July 10.
Die Schienenwanderung und Ihre Verhütung.* Alfred Wirth. (53) Serial beginning May 14.
Der Bau des Simplontunnels.* Konrad Pressel. (53) Serial beginning June 11.
Neuere Bauarten von Wechselstrom-Lokomotiven.* G. Brecht. (48) June 19.

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Electrical Equipment of the Washington Street Subway, Boston.* (17) May 22.
Labor Cost of Track Laying on Elevated Railways in New York City, also Some Costs of Erecting Steel.* (86) June 2.
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Eine Eisenbetonschwelle für Strassenbahnen.* Blass. (78) July 2.

Sanitary.

Sea Outfall-Sewers.* Ben Howorth. (63) Vol. 175.
The Discharge of Sewage into Tidal Waters. George A. Soper. (6) Apr.

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The Sewerage System of Salt Lake City.* L. H. Krebs. (Paper read before the Utah Soc. of Engrs.) (1) Apr.
Some Anomalies in Modern Plumbing Regulations.* J. Pickering Putnam. (Paper read before the Bost. Soc. Civ. Engrs.) (1) Apr.
Drain Tile and Drainage.* A. O. Anderson. (76) Serial beginning May.
Final Report of the Committee to Secure Data on Hot-Water Heating. (Amer. Soc. Heating and Ventilating Engrs.) (70) May.
Principles of Sewage Purification. George W. Fuller. (Lecture before Cornell Univ.) (14) Serial beginning May 15.
The Electrical Treatment of Sewage Effluent.* (73) May 21.
Treatment of Wool-Scouring and Dyeing Wastes at Dedham. (14) May 22.
The Practical Sterilisation of Water and of Sewage Effluents. H. C. H. Shenton. (Paper read before the Inst. of Mun. Engrs.) (96) Serial beginning May 28.
Air Cooling by Refrigeration.* W. W. Macon. (Paper read before the Amer. Soc. of Heating and Ventilating Engrs.) (70) June.
Waterbury Sewage and its Septic Action.* Wm. Gavin Taylor. (13) June 3; (14) June 5.
A Large Reinforced Concrete Sewer.* (14) June 5.
The Extension of the Glaise Creek Sewer in St. Louis.* (14) June 12.
A New Method of Treating Sewage.* K. Imhoff. (From *Surveyor and Municipal and County Engineer.*) (96) Serial beginning June 25.
The Disposal of Manufactural Wastes.* (14) June 26.
Septic Tank Patent Conference. (96) July 2.
The New Garbage Reduction Plant at St. Louis.* (14) July 3.
Modern Aspects of Sewage Disposal and Sewerage Problems. T. Aird Murray. (Paper read before the Saskatchewan Med. Assoc.) (96) July 16.
The Sewage Testing Station for the City of Philadelphia.* (14) July 17.
The Sanitary District of Chicago, and the Chicago Drainage Canal: A Review of 20 Years of Engineering Work.* Isham Randolph, M. Am. Soc. C. E. (13) July 22.
The Disposal of Manufacturing Wastes, Separately and with Domestic Sewage.* F. A. Barbour, M. Am. Soc. C. E. (Abstract of paper read before the Engrs.' Soc. of Pa.) (13) July 22.
Etudes sur l'Epuration des Eaux d'Egout sur des Lits Bactériens de Tourbe.* A. Müntz et E. Lainé. (92) Apr.
Note sur l'Incinération et sur les "Déstructeurs" d'Ordures Ménagère. E. Damour. (92) Apr.
Rafraichissement des Locaux Industriels par Evaporation d'Eau.* A. Papin. (33) May 1.
Beispiele aus dem Gebiete des Ventilatorenbaues.* Th. v. Bavier. (48) May 29.
Eisenbetonbauten für den Neubau der Kläranlage der Stadt Siegen.* Edwin Grahl. (78) June 10.
Die Leistungen von Kreiseln und Schöpfradern. H. Hazen. (48) Serial beginning July 3.
Uber Sammelkanäle und deren Höchstbeanspruchung.* Wilhelm Voit. (53) Serial beginning July 9.

Structural.

Portland Cement; the Le Chatelier Test for Soundness.* Arthur Charles Davis, Assoc. Inst. C. E. (63) Vol. 175.
Tests on Reinforced Concrete Beams.* E. Brown. (5) Vol. 22, Pt. 2.
The Resistance of Material to Impact.* T. E. Stanton. (75) Nov., 1908.
Foundation for the Building for the U. S. Naval Experiment Station at Annapolis, Md. Harrison W. Latta. (2) Jan.
Determination of the Effect of Age and Exposure upon the Tensile Strength of Portland Cement before Use.* Charles C. Phelps and others. (8) Apr.
The Development and Use of Steel Sheet Piling, with Some Data on the Preservation of Steel Buried in the Ground. J. R. Wemlinger. (A paper read before the Tech. League.) (86) May 19.
Storage Grain Elevator.* (15) May 21.
Essential Formulæ in Structural Engineering. Horace Cubitt. (Paper read before the Inst. of Mun. Engrs.) (96) May 28.
The Design and Construction of Industrial Buildings. Herbert F. Stimpson. (Lecture at Columbia Univ.) (14) Serial beginning May 29.
The Building and Equipment of the Keewatin Flour Mill.* (14) May 29.
Superstructure of the Liberty Street Ferryhouse, New York.* (14) May 29.
Protective Coatings for Structural Material. R. S. Perry. (4) June.
Steelwork in the La Salle Hotel, Chicago.* (14) June 5.
Queen-Post Roof Trusses.* Walter Hey and Thomas R. Sturgeon. (11) June 11.
Foundations of the Goldfield Consolidated Mill.* Percy E. Barbour. (16) June 12.
Raymond Concrete Pile Construction.* William F. Hall. (Paper read before the Technischer Verein.) (64) June 14.

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An Example of Defective Truss Construction.* C. A. P. Turner, M. Am. Soc. C. E. (13) June 17.
Methods of Constructing Concrete Silos with Some Data on Design.* (86) Serial beginning June 23.
The Setting of Portland Cement and its Regulation. H. K. G. Bamber. (Abstract of paper read before the Concrete Inst. of Great Britain.) (14) June 26.
The Rankine-Gordon and Euler's Formulæ for Columns.* W. E. Lilly. (12) July 2.
The Strength of Compression Members.* Edward Godfrey. (15) July 2.
The Mt. Clare Grain Elevator, Baltimore.* (14) July 3.
The Boston Safe Deposit & Trust Company's Building.* (14) July 3.
Discussion of Various Piles and Methods of Driving Them.* Frederic W. Swain. (Abstract from *Harvard Eng. Journal.*) (96) July 9.
Design of Steel Draft Stacks.* Birger F. Burman. (14) July 17.
The Mont Carmel Wing of the Chateau Frontenac, Quebec.* H. P. Borden. (14) July 24.
Materials and Methods of Construction for Industrial Buildings. J. P. H. Perry. (9) Aug.
Sur la Résistance au Flambage des Poteaux en Béton Armé.* Mesnager. (43) Mar.
La Construction en Pays de Tremblements de Terre.* G. Espitallier. (32) Mar.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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THE USE AND CARE OF THE CURRENT METER, AS PRACTICED BY THE UNITED STATES GEOLOGICAL SURVEY.*

By John C. Hoyt, Assoc. M. Am. Soc. C. E.

To be Presented September 15th, 1909.

Introduction.

A current meter for measuring the velocity of flowing water comprises two essential parts: (*a*) a wheel arranged so that when suspended in flowing water the pressure of the water against it causes it to revolve; (*b*) a device for recording or indicating the number of revolutions of this wheel. The relation between the velocity of the moving water and the revolutions of the wheel is determined by rating each meter.

The earliest type of meter was the float wheel, which was used by Borda and Dupuit in the latter part of the eighteenth century, and was practicable only for measuring velocities at the surface. About 1790, Woltmann modified this wheel so that it could be used beneath the surface, the number of revolutions being recorded by a gear mechanism, which was started and stopped at the beginning and end

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Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

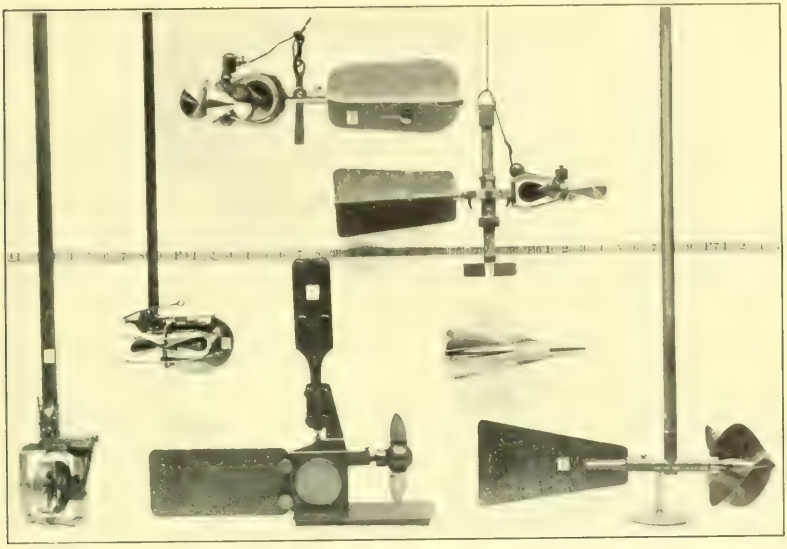


FIG. 1.—TYPES OF METERS EXPERIMENTED WITH BY U. S. GEOLOGICAL SURVEY.

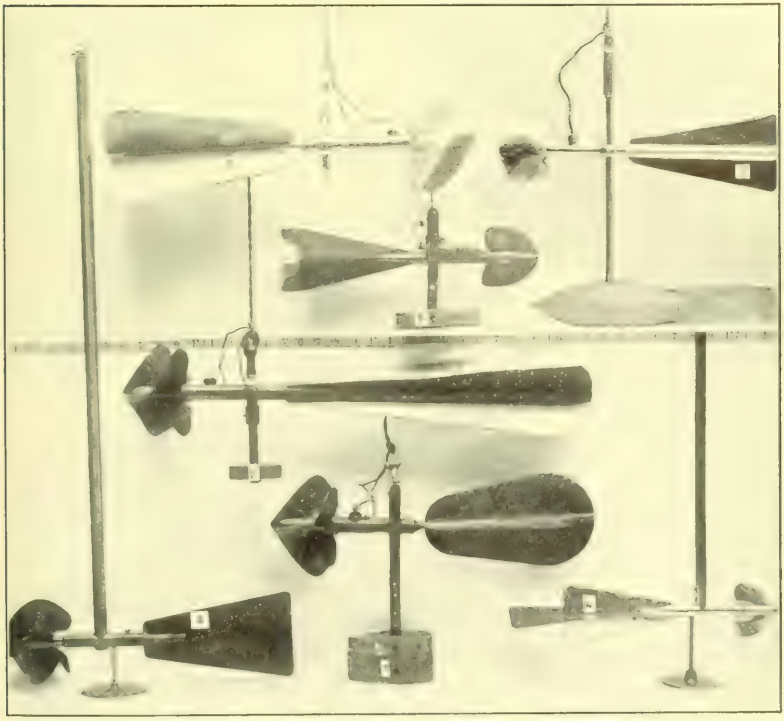


FIG. 2. VARIOUS FORMS OF THE HASKELL METER.

of a run by a catch operated by a cord. It was necessary, however, to lift the meter out of the water in order to read it. LaPointe arranged the recording apparatus above the surface by connecting the axle with a vertical rod and beveled gear; and Baumgarten, Saxton, Brewster, Laignel, and others made various modifications of the instrument. Prior to the invention of the electric recorder for indicating the number of revolutions of the wheel, users of the meter always experienced difficulty caused by the resistance due to the recording mechanism and its tendency to get out of order.

In America current meters were earliest used in connection with the investigations of the Mississippi, started in 1850 by Humphreys and Abbot,* when the ship's log and the Saxton meter were used to a small extent and with little success.

About 1860, the late D. Farrand Henry, M. Am. Soc. C. E., Assistant on the United States Lake Survey, invented for use with the current meter an electrical recorder,† which eliminated the frictional and the other difficulties of the mechanical recorder, and made feasible the further development of the meter.

The first extended and successful series of measurements with the current meter in the United States was made on the Connecticut River by the late T. G. Ellis, M. Am. Soc. C. E., in connection with studies begun in 1871.‡ General Ellis started his work with the Woltmann meter, equipped with an electrical recording device, but later used an electrical recording meter devised by himself. The results obtained by these measurements have had an important effect on the development of stream-gauging instruments.

The earliest American patents for current meters were taken out in 1851. There are now on file in the Patent Office, classified under ship's logs, more than fifty patents for devices for measuring the velocity of water, and many other devices have been constructed but not patented. The only ones which have had much general use, however, are the Price, Haskell, Fteley, and Ellis meters (Plates XXI, XXII, and XXIII, and Fig. 1).

Each of the various meters has been developed to meet the requirements of some special condition, and, until recently, the use of all

* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

† *Journal* of the Franklin Institute, Vol. XCII, 1871.

‡ Report, Chief of Engineers, U. S. Army, 1878, Part I.

has been confined to special hydrographic investigations in connection with some public work, municipal, State, or Federal. The present widespread interest in hydraulic development has created such a demand for measurements of the flow of streams that the current meter is now in general use, and is essential to the equipment of the engineer engaged in hydraulics.

In 1888 the United States Geological Survey began the gauging of streams. The plans called for measurements in all sections of the country, and the conditions to be provided for presented an infinite variety in combinations of range in depth, width, and velocity. No adequate meter was available for work of this extended nature, and, furthermore, elaborate equipment and methods were prohibited by limited funds. It was necessary to devise or adapt an instrument which could be readily carried in the field and operated by one man, either from a bridge, a boat, a cable and car, or by wading.

After experimenting with various types (Plates XXI and XXII), the engineers of the Survey developed a meter combining the essential features of the Price acoustic and large Price electric meter (Plate XXII). This is known as the small Price meter, and has been in general use in the Survey work. Modifications in its construction have been made from time to time, based on the experience of various engineers, until now it represents the experience of more than twenty years of stream gauging (Plate XXIII).

The methods developed by the Survey engineers are believed to represent the best practice in this line of work. The Survey's data are now used extensively in hydraulic development in all parts of the country, and its methods have been adopted in similar work by many engineers in all parts of the world. A description of the instruments and their use, therefore, should be of interest to all engineers.

General Features of Current Meters.

Current meters, in general, may be divided into two classes: direct action and differential action, the division depending on whether the water, in revolving the wheel, does or does not exert a force which tends to retard the motion of the wheel.

The wheels of the direct-action meters consist of flat or warped-surface vanes, set on a horizontal axis, which revolve by the direct pressure of the water against the blades. In a meter of this type the

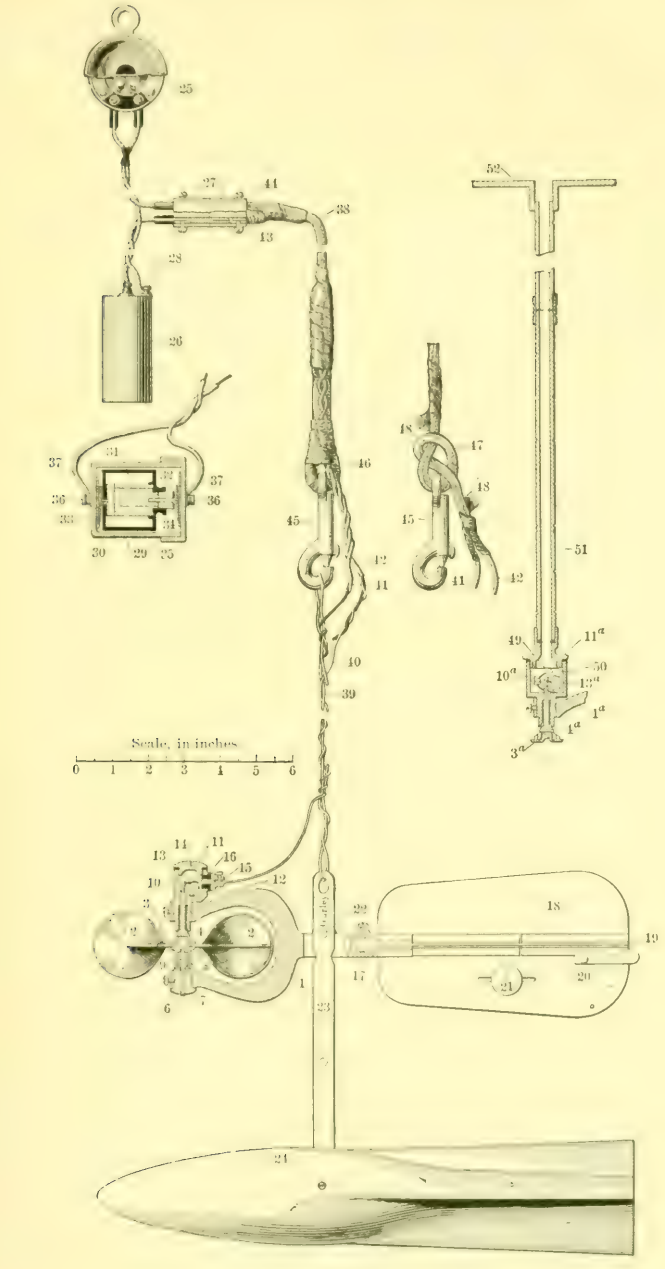


FIG. 1.

friction increases as the velocity decreases. The revolutions of the wheel are quite rapid for high velocities, but the resistance due to the area of the cross-section of the meter against the water is small. The principal direct-action meters are the Haskell and the Fteley (Plate XXI).

The wheels of the differential meters are made up of a series of cups which revolve on a vertical axis. The pressure of the water on the cups, being less on the convex than on the concave side, causes the wheels to revolve. In a meter of this type, the friction increases with the velocity, but the increased frictional effect is overcome by the increased motive power due to the velocity. The area of the cross-section offers a larger resistance to the current than in the direct-action meter, but the wheel revolves more slowly owing to the retarding motion caused by the resistance on the convex side of the cups. The principal types of differential meters are the Price and the Ellis (Plates XXI and XXII).

The essentials for a good current meter are (*a*) simplicity in construction, with no delicate parts which easily get out of order; (*b*) a small area of resistance to the velocity of the water; (*c*) a simple and effective device for indicating the number of revolutions of the wheel; and (*d*) easy adaptability for use under all conditions.

Fig. 1 and Plates XXIV and XXV illustrate the difference in construction between meters of American and foreign make. The foreign meters are much more delicately constructed than the American, and their mechanism is more complicated, so that several men are needed to make a discharge measurement (Fig. 1, Plate XXVI) instead of one man, as with American meters (Plate XXVII).

Although the small Price meter is the only one described herein, an effort has been made to adapt the general discussion to the use of current meters of any make.

Description of the Small Price Current Meter and Equipment.

The small Price current meter (Fig. 1) and equipment consists of five principal parts: (1) the head; (2) the tail; (3) the hanger and weights; (4) the recording device; and (5) the suspending device. In the following descriptions the numbers in parentheses refer to those on Fig. 1.

The Head.—The head consists of a ⊂-shaped yoke (1) carrying

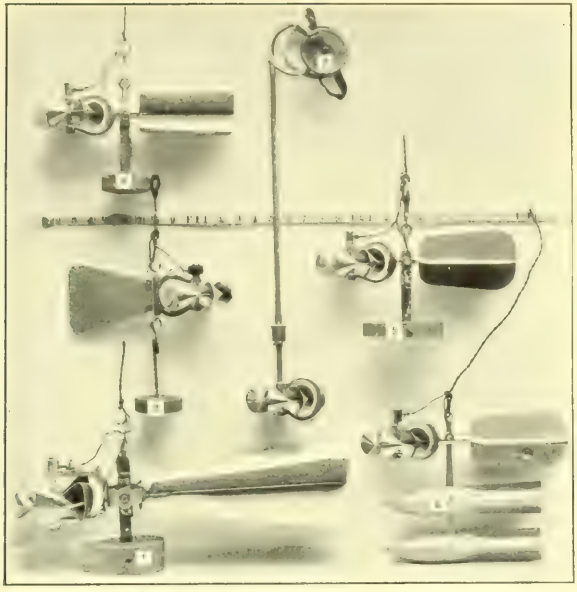


FIG. 1. VARIOUS FORMS OF RECORDING DEVICES.

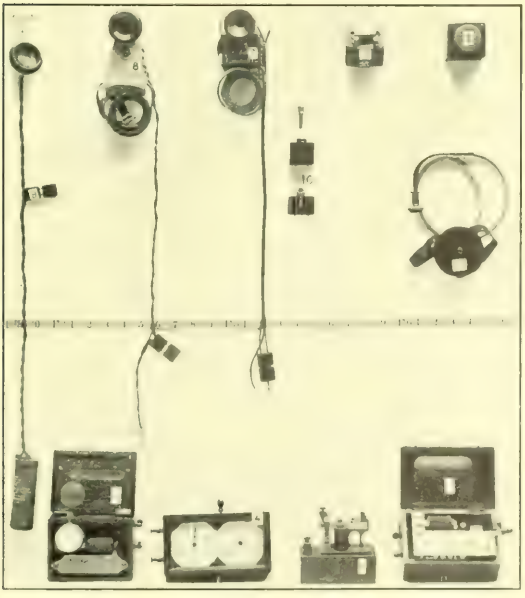


FIG. 2. VARIOUS FORMS OF THE PRICE METER.

a wheel made of six conical cups (2), brazed to a horizontal frame (3). This wheel, referred to as the cups, turns on a vertical axis known as the cup shaft, which sets on a cone point bearing at the lower end and engages the recording mechanism at the upper end.

The cup shaft consists of two parts (4 and 5), screwed together from either side of the cup frame, thus fastening the cups rigidly to the cup shaft. At the lower part of the cup shaft there is a cone bearing which receives the cone point (6) on which the cups revolve. The cone point is screwed through a metal bushing (7) known as the cone plug, and is firmly held by a lock-nut (8). The cone plug fits into the lower arm of the yoke by a sliding connection, and is clamped in position by a set-screw. By means of a sleeve-nut (9) on the lower part of the shaft, the cups can be lifted from the cone point when the meter is not in use. This sleeve-nut has a left-handed thread, so that it will not tighten when the cups revolve. The upper part of the cup shaft is fitted with a worm gear which passes into a cylindrical chamber (10 or 10ª), known as the contact chamber, as it contains the mechanism for making the contact which indicates the revolutions of the cups. The construction and arrangement of both the contact chamber and the mechanism contained in it depend on whether an acoustic or an electric recording device is used.

When the electric recording device is used, the contact chamber (10) is closed by a screw cap (11) provided with a leather gasket for keeping out the water. It fits by a sliding connection into the upper end of the yoke, and is clamped into position by a set-screw. In the contact chamber there is fitted a cylindrical plug (12) which is held in position by a screw and carries a gear-wheel (13) which engages a worm gear on the upper end of the cup shaft, the gearing being arranged so that the wheel makes one revolution for every twenty revolutions of the cups. On the side of the wheel there are four platinum pins, equally spaced and set so that they will strike the contact spring (14) at each fifth revolution of the cups, thus closing the electric circuit, which is indicated by the recording device, as explained later. These contact parts are known as the contact wheel, the contact pins, and the contact spring. The contact spring is of platinum, and is carried by the contact plug (15) which is screwed into the contact chamber through a hard rubber bushing (16), thus insulating the contact spring from the meter when it is not touching

one of the pins on the contact wheel. In the end of the contact plug there is a hole and a set-screw for connecting with a wire from the recording device.

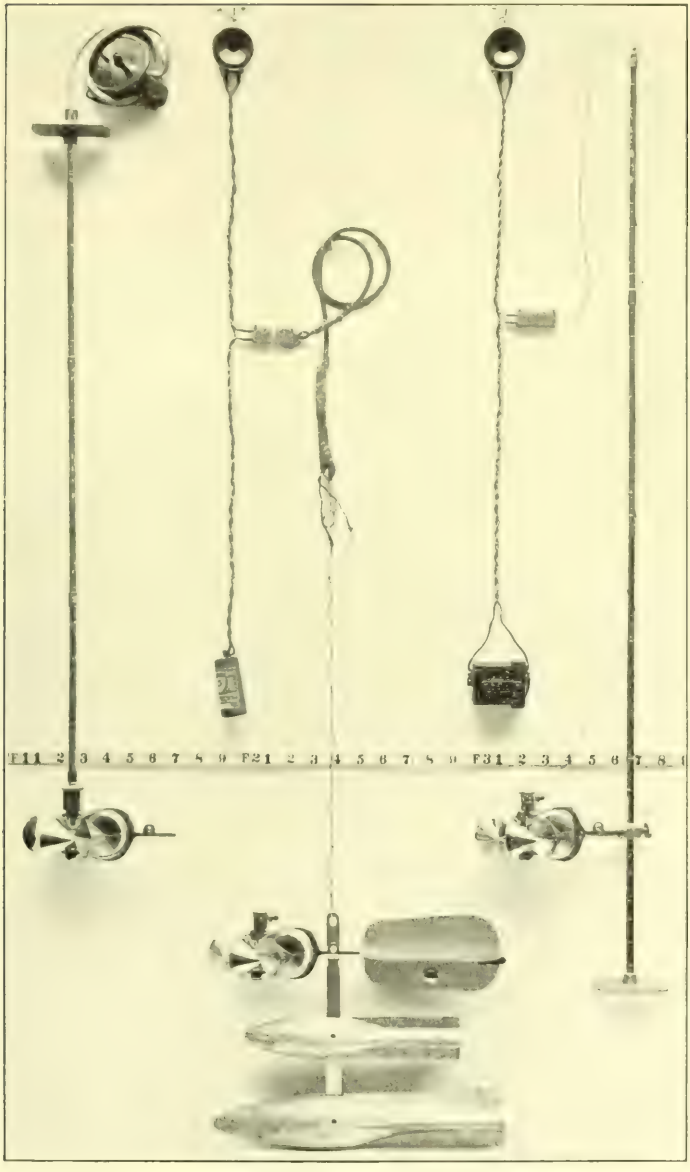
When the acoustic recording device is used, the contact chamber (10^a) is closed with a cap (11^a) and a metal drum (49), and, in place of the platinum contact spring (14) and the plug (16), there is a small hammer (50) which is arranged so that it is thrown against the drum by the pins on the side of the gear-wheel (13^a) at each fifth revolution of the cups.

The stem of the yoke contains a slot and a screw hole (22) for attaching the meter hanger (23), and a socket into which the stem of the tail of the meter (17) is fastened.

The Tail.—The tail is used when the meter is suspended by a cable. It provides for balancing the head, and also keeps the axis of the meter parallel to the direction of the current. It consists of a stem (17) which fits by a sliding connection into the socket in the stem of the yoke where it is clamped by a set-screw. On this stem there are two vanes (18 and 19) set at right angles. One of the vanes is rigidly attached to the stem; the other fits into it by grooves, so that it can readily be pulled out when the key (20) which holds it in place is turned. On one of the vanes there is a slot carrying a weight (21) which can be adjusted so as to balance the meter.

The Hanger and Weights.—When suspended with a cable, the meter is hung by a screw-bolt (22) on a steel stem (23) which passes through a slot in the stem of the yoke. The slot in the stem of the yoke is wide enough to allow the meter to swing freely in a vertical plane, and the bolt passes through the frame a little above the center of gravity of the meter, so that the latter will readily adjust itself to a horizontal position. In the upper end of the hanger there is a hole for attaching the suspending cable, and at intervals along the stem there are other holes by which the meter and lead weights may be hung. The weights (24) are of torpedo shape—this design offering the least resistance to the current—and are made in two sizes, weighing, respectively, 6½ and 15 lb. They are attached to the stem by a screw bolt. The manner of arrangement of the weights and meter on the stem depends on the conditions under which the measurements are to be made.

When the meter is used on a rod, the tail, hanger, and weight are



PRICE METERS AS NOW USED.

dispensed with, and the head is held by a sliding carrier (Plate XXIII), or it is attached to the rod (51) by a screw connection on the contact chamber when the acoustic recording device is used.

The set-screws for clamping the various sliding connections are all of the same size and of standard make. Beveled grooves are provided in each of these connections so that when the set-screws engage them the parts are drawn into place.

All parts of the meter are standard, and can readily be replaced in the field.

The Recording Device.—A recording or indicating device is necessary for determining the number of revolutions of the meter wheel, and the successful use of the meter depends primarily on this part of the apparatus.

Various devices, Fig. 1, Plate XXII, operated either on the mechanical, electric, or acoustic principle, have been used for this purpose. These include the telegraph ticker, electric buzzer, telephone receiver, drums, etc. Of these, however, the telephone attachment and the acoustic recorder have been found to be most satisfactory in general practice.

The telephone attachment, Fig. 1, consists of a telephone receiver (25) and small battery (26), connected in a circuit which terminates in a connecting plug (27) by which the apparatus can be readily connected or disconnected in circuit with the meter, depending on the method of suspension, as explained later.

The magnets of the telephone receiver are wound for 10-ohm resistance so as to make as loud a click as possible.

Either a dry-cell or a wet-cell battery may be used. The most satisfactory dry cell (26) is the No. N 118, "Every Ready" cell, which is 1 in. in diameter and 3 in. long. Connection is made with this cell through two screw connecting posts (28), both at the same end of the cell.

The wet cell consists of an outer casing of hard rubber (29), about 1½ in. square, containing a carbon compartment (30) into which a zinc pole (31) having a rubber stopper (32) is inserted. The current is generated by a solution of bisulphate of mercury and water. Contact is made from the cell through a platinum plug (33) extending into the carbon at the bottom and through the screw (34) in the zinc pole which extends through the rubber stopper.

The cell is encased in a leather box (35), and connection is made

with it through two screw connecting posts (36), each of which terminates in a separate spring plate (37) against which the poles of the battery bear.

In use, the receiver is pinned to the left shoulder and the battery cell is placed in the side coat pocket. The connecting plug (27) will then hang a little below the shoulder and is easily accessible for attaching and detaching the meter.

In the acoustic recorder, the striking of the hammer (50) on the drum (49) in the contact chamber (10^a) indicates each fifth revolution of the meter, as already explained. The sound is transmitted through the rods (51) and a rubber tube to the ear of the observer. The rubber tube and ear-piece (Plate XXIII) are not necessary unless there is considerable noise at the station, as the click is loud enough to be heard without them.

Automatic recorders, Fig. 1, Plate XXII, have been used to some extent to determine the number of revolutions of the meter. For general work, however, these are not entirely satisfactory, as they are likely to get out of order, and are inconvenient to carry.

The time taken by the meter wheel to make a given number of revolutions should be determined with a stop-watch, as by this means the error in the observation can be reduced by observing to a fractional part of a second.

The Suspending Device.—The suspending device, which consists of a rod or of some form of cable, must make provision for lowering the meter and weight into the water and also for completing an electric circuit between the meter wheel and the recording device.

The rod in common use in connection with the electric recorder consists of a ½-in. tube (Plate XXIII) graduated to feet and tenths. For convenience in carrying, it is made in 1½-ft. sections which screw together. On the bottom of the rod there is a flat foot which keeps it from sinking into the bottom of the stream, and at the top there is a plug for connecting one of the wires from the recording device. The head of the meter is attached to a sliding hanger, which can be moved up and down the rod or clamped in any position. The circuit between the meter wheel and the recording device is made by attaching one of the wires from the recording device to the plug in the top of the rod. The other wire follows down the rod, and is attached to the contact plug of the meter.

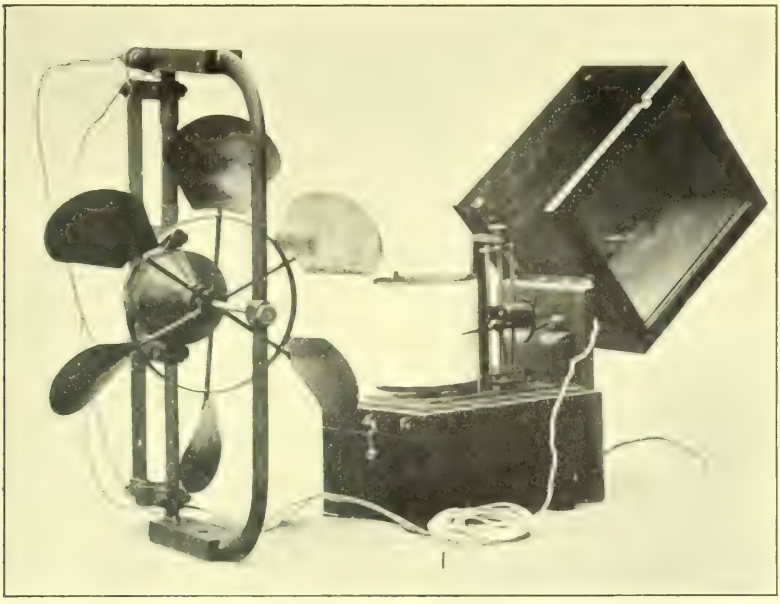


FIG. 1.—RICHARDS METER, WITH ELECTRICAL RECORDER.

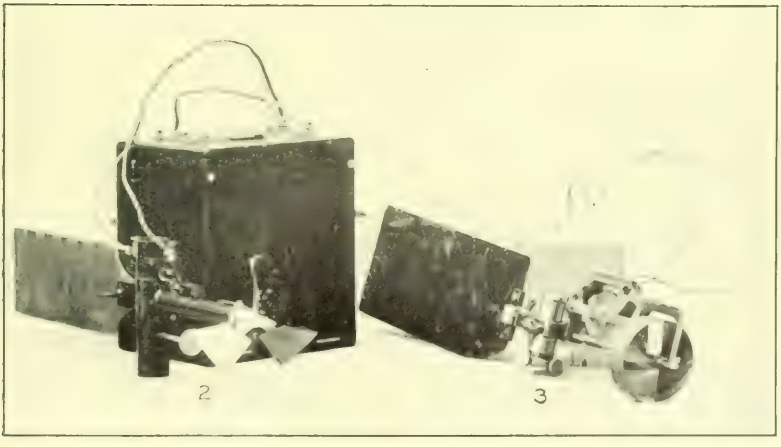


FIG. 2.—OTT AND WOLTMAN METERS, WITH ELECTRICAL RECORDERS.

In using the meter on the rod, the foot-piece of the rod is set on the bottom of the stream, and the head of the meter is adjusted so that it will come to the point at which it is desired to measure the velocity. By this means the meter can be held firmly in the required position.

The rods (Plate XXIII) for use with the acoustic recorder are of ½-in. tubing graduated to feet and tenths, and, for convenience in carrying, are made in 1-ft. sections which screw together. The bottom rod connects with the contact chamber with a screw, and is cut so that the distance from the center of the cups to the end of the rod is just 1 ft. On the upper end of the top rod there is a flat plate, in the center of which there is a hole through which the sound from the drum can be heard. The soundings are made with this end of the rod, and the plate keeps the end from sinking into the bed of the stream.

The best form of cable (Fig. 1) is a combination of No. 16, old code, double-insulated, show-window cord (38) and No. 12 or 14 galvanized wire (39) about which is wound a small insulated magnet wire (40).

The show-window cord is used for the upper part of the cable. It is large enough to be manipulated easily with bare hands, and, being made of two insulated wires, provides for making the direct and return circuit between the meter and the recording device. In its use, the two wires of which it is made must be separated at either end (41, 42, 43, 44) in order to make the attachment with the connecting plug (27) of the recording device at the upper end and with the galvanized wire (39) and magnet wire (40) which lead to the meter at the lower end. A ring or snap (45), into which the galvanized wire is looped, is fastened, either by a loop (46) or a knot (47), to the lower end.

In fastening the meter cable to the snap with a knot (47), a strip of adhesive tape is wound around the cable two or three times, about 1 ft. from the end, leaving about 6 in. of the tape at the beginning and end of the winding. The cable is then inserted through the snap or ring so that the snap bears on the adhesive tape, and a knot is tied in the cable about the snap (45) and drawn down as tight as possible. The ends of the adhesive tape (48) are then wound around the cable, one above and the other below the knot, to keep it from sliding. The outside covering of the cable can then be taken off

within 3 or 4 in. of the knot, and one wire fastened to the wires leading to the contact plug and to the hanger.

If the snap is held in a loop (46), a length of about 12 or 14 in. of the outside insulation is removed so that the wires can be doubled back and connected with those leading to the contact plug and hanger. The loop is first tied with string and then wound with adhesive tape, the tape being placed around the cable where the ring bears on it.

The galvanized and magnet wires (39 and 40) make up the lower end of the cable, and should be long enough to reach from the surface to the deepest point in the stream. Their use is advantageous because they offer small resistance to the moving water and thus prevent the meter from being carried down stream.

The galvanized wire (39) provides both for carrying the weight of the meter and conducting the electric current between the meter and the recording device. It is attached by ordinary loop connections to the snap in the lower end of the show-window cord and to the meter hanger (23). The direct circuit is made through it by the direct connection with the meter stem and by its connection at the upper end with one of the insulated wires (41) from the show-window cord.

The fine magnet wire (40), which provides for the return circuit between the meter and the recording device, is wound loosely around the galvanized wire, and may be fastened with tire tape. At the upper end it is connected with one of the insulated wires (42) from the show-window cord, and at the lower end with the contact plug (15) of the meter. In order to aid in preserving the insulation between the galvanized and magnet wires they should be shellacked; and, even with this precaution, it may be necessary to replace the magnet wire frequently.

If the velocities are not so great as to carry the meter down stream, the galvanized and fine magnet wire may be dispensed with. The snap (45) at the lower end of the show-window cord may be attached directly to the meter stem, the circuit being completed by attaching the insulated wires (41, 42) to the contact plug and to the screw on the meter hanger, respectively.

The meter may also be suspended by a single uninsulated galvanized wire, the return circuit being made through the ground and water. In using the single wire the connection is from the water to the meter through the contact point to the line, then to the battery and through the 'phone to the bridge or cable, then to the ground and back

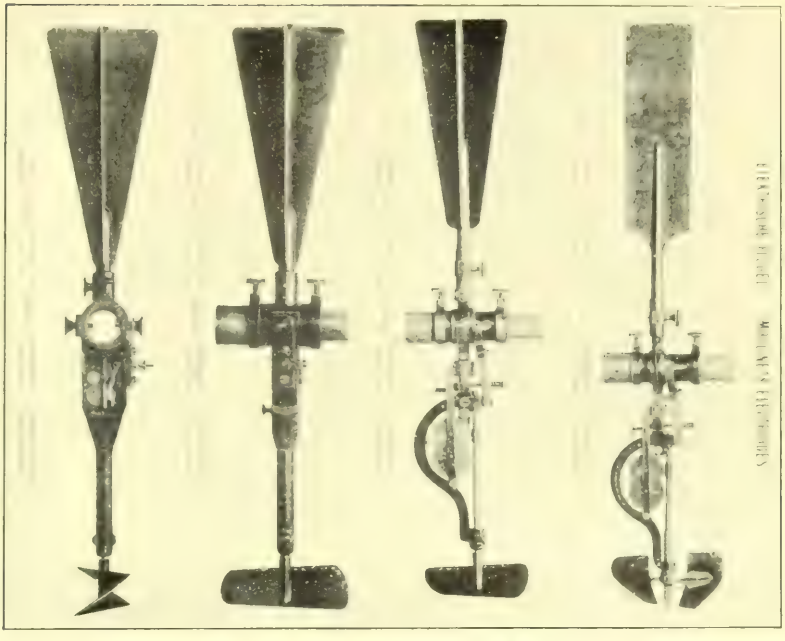


FIG. 1.—TYPES OF AMSLER METER USED IN THE SWISS HYDROGRAPHIC SERVICE.

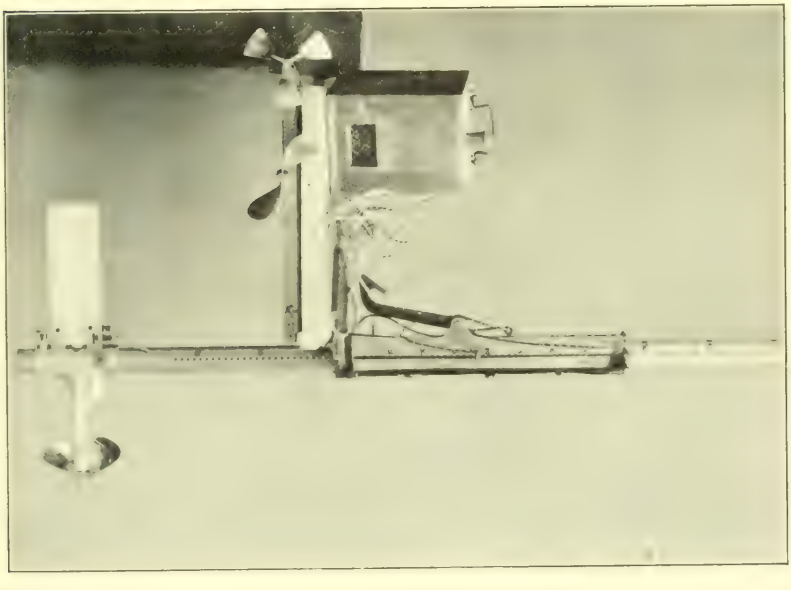


FIG. 2.—OTT METER USED IN THE FRENCH HYDROGRAPHIC SERVICE.

to the water. It does not make any difference on which side of the battery the 'phone is placed in the line (Fig. 2).

When using a single wire, on a cable or a bridge, a clean metallic contact should be made. A little paint, rust, or other coating will prevent good work.

In measuring high velocities and deep streams, where the suspending cables afford insufficient support, stay-lines or guy-lines are used to keep the meter in place.

Rating the Meter.

The relation between the revolutions of the meter wheel and the velocity of the water must be determined by rating each meter before it is used. Theoretically, the rating for all meters of the same make should be the same, but, as a result of slight variations in construction, and in the bearing of the wheel on the axis at different velocities, the ratings differ.

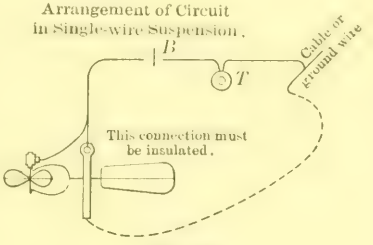


Fig. 2.

A meter is rated by conducting it through still water with uniform speed, and noting the time, the number of revolutions, and the distance. The revolutions per second and the velocity in feet per second are afterward computed from these data. Many runs are made, as shown in Table 1, the speeds varying from the least which will cause the wheel to revolve to several feet per second. The results of these runs, when plotted (Fig. 3) with revolutions per second and velocity in feet per second as co-ordinates, locate the points which define the meter rating curve from which the rating table is prepared, as described later.

The United States Geological Survey has three rating stations, one at Washington, D. C., one at Denver, Colo., and one at Los Angeles, Cal. (Fig. 2, Plate XXVI). These stations consist of a platform over the water carrying a track about 200 ft. long, on which a car for carrying the meter is run. The track is near the edge of the platform, and the meter is suspended in the water by a projecting arm. At the Denver and Washington stations the car is propelled by hand; at the Los Angeles station it is moved electrically.

The number of revolutions of the meter wheel are indicated on an electric recorder; the distance is obtained by an electrical mechanism, which is in circuit with the meter wheel, so that the exact distance for a given number of revolutions is obtained; and the time is taken by a stop-watch.

TABLE 1.—Observations for Small Price Penta Meter Rating. Made December 14th, 1908, at Chevy Chase Lake, Maryland, by Henshaw and Walters. Meter Hung with one 15-lb. and one 6½-lb. Torpedo Weight.

No. of run.	Observations for Length of Run.			Time, in seconds.	No. of revolutions.	Revolutions per second.	Velocity, in feet per second.	Remarks.
	Start.	End.	Distance.					
	Feet.	Feet.	Feet.					
1	−6.8	32.0	25.2	79	10	0.127	0.319	
2	21.4	+2.9	24.3	65	10	0.154	0.374	
3	−7.3	32.1	24.8	67	10	0.149	0.370	
4	25.0	−0.4	24.6	62	10	0.161	0.397	
5	−10.7	35.2	24.5	45	10	0.222	0.544	
6	28.2	−2.8	25.4	50	10	0.200	0.508	
7	−13.6	62.5	48.9	78	20	0.256	0.627	
8	54.2	−5.1	49.1	77	20	0.260	0.638	
9	−17.1	66.6	49.5	64	20	0.312	0.773	
10	61.4	−12.9	48.5	68	20	0.294	0.713	
11	8.7	56.2	47.5	48	20	0.417	0.990	
12	52.1	−5.0	47.1	50	20	0.400	0.942	
13	−1.2	96.4	95.2	76	40	0.526	1.252	
14	83.7	+8.7	92.4	72	40	0.556	1.283	
15	+10.0	82.9	92.9	47	40	0.851	1.977	
16	90.0	+1.8	91.8	50	40	0.800	1.836	
17	+2.0	91.6	93.6	35	40	1.14	2.68	
18	90.7	−0.3	90.4	29	40	1.38	3.12	
19	−9.7	100.8	91.1	14·8	40	2.70	6.16	
20	95.9	−6.3	89.6	17·4	40	2.30	5.15	
21	−4.6	95.6	91.0	12·8	40	3.12	7.11	
22	97.4	−6.8	90.6	11·6	40	3.45	7.81	
23	−10.0	101.3	91.3	11·2	40	3.57	8.15	
24	83.7	+7.4	91.1	9·2	40	4.35	9.90	
25	0.0	92.7	92.7	9.0	40	4.44	10.30	
26	98.0	−7.7	90.3	9.2	40	4.35	9 82	
27	+7.5	86.4	93.9	8.8	40	4.55	10.67	
28	94.5	−1.6	92.9	7.8	40	5.13	11.91	
29	−1.2	93.9	92.7	7.4	40	5.41	12.53	
30	100.6	−9.1	91.5	7.8	40	5.13	11.73	
31	+2.4	91.6	94.0	7.0	40	5.71	13.43	
32	106.0	−13.0	93.0	7.2	40	5.56	12.92	
33	+5.6	88.4	94.0	6.2	40	6.45	15.16	
34	101.1	−7.9	93.2	6.6	40	6.06	14.12	

Note.—The runs are in pairs, the odd numbers being across the track and the even numbers in the return to the starting point.

Theoretically, the wheel of a differential-action meter, when carried through still water, should revolve as a wheel revolves in passing over the ground. That is, in going a given distance it should make practically the same number of revolutions, regardless of speed. The rating

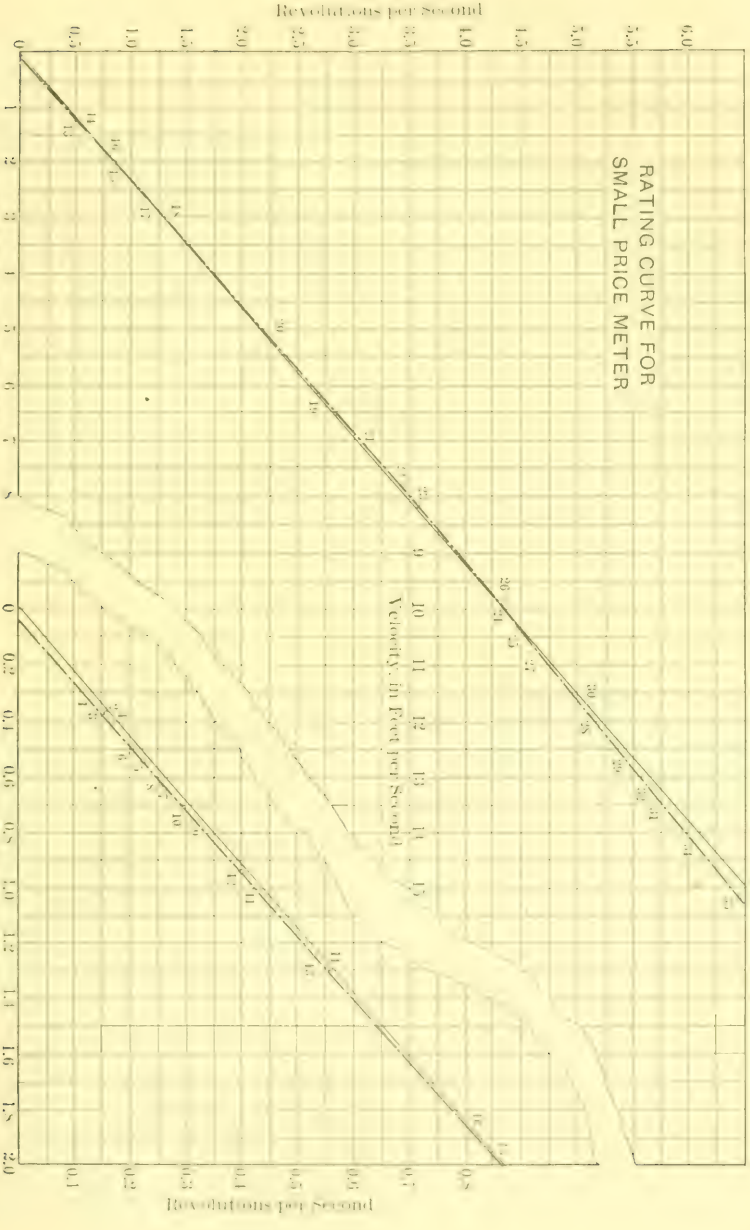

RATING CURVE FOR
SMALL PRICE METER
Revolutions per Second
Velocity, in Feet per Second
Velocity, in Feet per Second
Revolutions per Second

Fig. 3.

of a great many small Price electric meters shows this number to be from 42 to 44 revolutions in going 100 ft.

The true number of revolutions of the wheel should equal the distance of the run divided by the effective circumference of the wheel multiplied by a coefficient which depends on the retarding effect due to the pressure on the convex surface of the cups and the blanketing effect of the cups. Assuming the effective circumference to be the circle passing through the points of the cups, which is 0.7854 ft., and the true number of revolutions to be 43½ per 100 ft. run; then the coefficient would be 0.345. Although complete data are not available to confirm this theory, the working of the meter shows that it holds very closely to it.

The foregoing shows that the theoretical meter-rating curve is a straight line passing through the origin. If the true number of revolutions made in going 100 ft. is 43½, the equation of this curve will be $X = 2.3\ Y$, where X = velocity, in feet per second, and Y = revolutions per second.

It is interesting to note that Dubuat found that the resistance of a circular plate, when drawn through still water, as compared with that of a sphere of the same diameter, was as 100 to 35.

A study of the rating curves of a large number of small Price meters shows that, as a rule, the curve is made up of two straight lines, the extension of the lower one joining the upper one in an angle between the velocities of 8 and 9 ft. At this point there is a slight increase in the friction on the bearings of the meter wheel and shaft. Notwithstanding this break in the curve, the observed curve parallels the theoretical curve very closely. The lower part of the curve starts at a velocity of less than 0.1 ft. per sec., which is required to start the wheel.

In plotting the rating curve, velocities in feet per second are generally shown as abscissas and revolutions per second as ordinates. The scale for velocities of less than 2 ft. per sec. should be 5 in. per ft. of velocity, and, for greater velocities, 1 in. per ft. of velocity. The scale for revolutions per second, for velocities of less than 2 ft., should be 10 in. per rev. per sec., and for velocities greater than 2 ft., it should be 2 in. per rev. per sec. This necessitates the construction of two curves: one for low velocities and one for high velocities. The low-velocity curve should be constructed first and two points taken from

FIG. 1.—MEASUREMENTS AT BOAT GAUGING STATION, ON RIVER DURONÉE, NEAR ROUSSET, FRANCE.

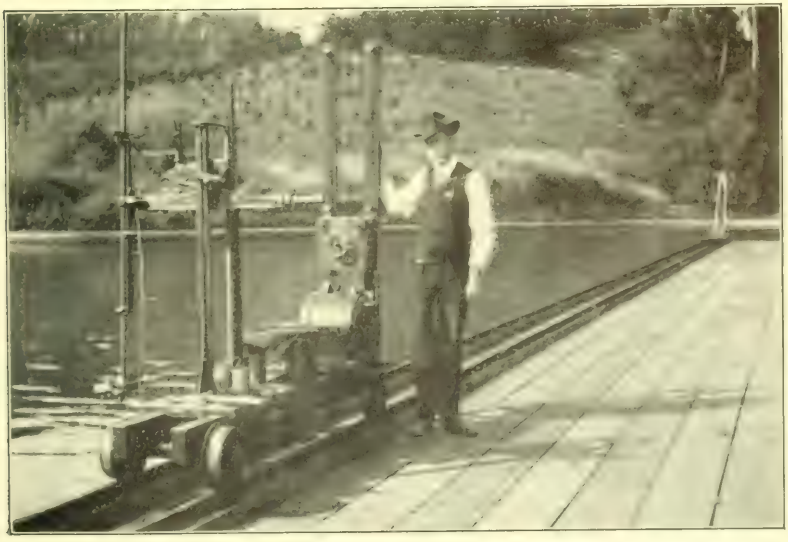


FIG. 2.—U. S. GEOLOGICAL SURVEY CURRENT-METER RATING STATION, LOS ANGELES, CAL.

this curve should be replotted at the smaller scale, in order to determine the position of the lower portion of the curve. The theoretical, or a mean, rating curve should also be drawn on the sheet as a guide for drawing the curve.

In using the meter, observation is made of the number of seconds the wheel requires to make a given number of revolutions. Therefore, a rating table is prepared for each meter, giving the velocities per second corresponding to the number of revolutions. The most convenient table is prepared for 5, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 150, and 200 revolutions, with the times of the runs ranging from 30 to 60 sec., giving velocities from 0.234 to 15.69 ft. per sec.

The rating table (Table 2) is prepared by tabulating the number of revolutions per second from 30 to 60 sec. for each length of run, as shown in the column headed "Revolutions per second." These values enable taking from the curve, Fig. 3, the velocity per second for each number of seconds run.

As the curves for various meters are practically parallel, differences for consecutive velocities on each are practically the same. Therefore, it is only necessary to take occasional velocities from the curve, and intermediate values can be filled in by adding the successive differences from a mean table, as given in the column headed "Difference." These differences also aid in using the table to interpolate for fractions of a second.

In preparing the mean rating curve and table from which the values in the column headed "Difference" have been obtained, the mean of the ratings of nine meters was taken, the rating table, illustrated by Tables 1 and 2 and Fig. 3, is for a current meter having a rating which is identical with this mean curve and table. Such coincidences often appear, either for the whole curve or for a portion of it.

Table 3 gives the percentage of deviation from the mean curve of the different curves used in determining it. An examination of this table shows that the error introduced, due to differences in the ratings of the meters, is very small, and that for meters in good condition the standard rating table can be used without serious error for all velocities greater than 0.5 ft. per sec. The comparison of numerous other rating curves with the standard shows practically the same percentage of variation.

TABLE 2—*Continued.*

41	0.122	0.322	0.007	0.244	0.599	0.013	0.488	1.15	0.03	0.732	1.71	0.04	0.976	2.26	0.05	1.22	2.82	0.07	2.44	5.60	0.13	41
42	0.119	0.315	0.006	0.238	0.586	0.013	0.476	1.13	0.03	0.714	1.67	0.04	0.952	2.21	0.05	1.19	2.75	0.06	2.38	5.46	0.13	42
43	0.116	0.309	0.006	0.233	0.573	0.012	0.465	1.10	0.02	0.698	1.63	0.04	0.930	2.16	0.05	1.16	2.69	0.06	2.33	5.34	0.12	43
44	0.114	0.303	0.006	0.227	0.561	0.011	0.455	1.08	0.02	0.682	1.60	0.03	0.909	2.11	0.05	1.14	2.63	0.06	2.27	5.22	0.11	44
45	0.111	0.297	0.005	0.222	0.550	0.011	0.444	1.06	0.02	0.667	1.56	0.03	0.889	2.07	0.04	1.11	2.57	0.05	2.22	5.10	0.11	45
46	0.109	0.291	0.005	0.217	0.539	0.011	0.435	1.03	0.02	0.652	1.53	0.03	0.870	2.02	0.04	1.09	2.52	0.05	2.17	4.99	0.11	46
47	0.106	0.286	0.005	0.213	0.528	0.010	0.426	1.01	0.020	0.638	1.50	0.03	0.851	1.98	0.04	1.06	2.47	0.05	2.13	4.89	0.10	47
48	0.104	0.281	0.005	0.208	0.518	0.010	0.417	0.992	0.019	0.625	1.47	0.03	0.833	1.94	0.04	1.04	2.41	0.05	2.08	4.79	0.10	48
49	0.102	0.276	0.005	0.204	0.508	0.009	0.408	0.973	0.019	0.612	1.44	0.03	0.816	1.90	0.04	1.02	2.37	0.05	2.04	4.69	0.09	59
50	0.100	0.272	0.004	0.200	0.499	0.009	0.400	0.954	0.018	0.600	1.41	0.03	0.800	1.86	0.04	1.00	2.32	0.04	2.00	4.60	0.09	50
51	0.098	0.267	0.004	0.196	0.490	0.009	0.392	0.937	0.017	0.588	1.38	0.03	0.784	1.83	0.03	0.980	2.28	0.04	1.96	4.51	0.09	51
52	0.096	0.263	0.004	0.192	0.482	0.008	0.385	0.919	0.017	0.577	1.36	0.02	0.769	1.79	0.03	0.962	2.23	0.04	1.92	4.42	0.08	52
53	0.094	0.259	0.004	0.189	0.473	0.008	0.377	0.903	0.016	0.566	1.33	0.02	0.755	1.76	0.03	0.943	2.19	0.04	1.89	4.34	0.08	53
54	0.093	0.255	0.004	0.185	0.465	0.008	0.370	0.887	0.015	0.556	1.31	0.02	0.741	1.73	0.03	0.926	2.15	0.04	1.85	4.26	0.08	54
55	0.091	0.251	0.004	0.182	0.458	0.007	0.364	0.872	0.015	0.545	1.29	0.02	0.727	1.70	0.03	0.909	2.11	0.04	1.82	4.18	0.07	55
56	0.089	0.247	0.004	0.179	0.450	0.007	0.357	0.857	0.014	0.536	1.26	0.02	0.714	1.67	0.03	0.893	2.08	0.04	1.79	4.11	0.07	56
57	0.088	0.244	0.003	0.175	0.448	0.007	0.351	0.843	0.014	0.526	1.24	0.02	0.702	1.64	0.03	0.877	2.04	0.03	1.75	4.04	0.07	57
58	0.086	0.240	0.003	0.172	0.436	0.007	0.345	0.829	0.013	0.517	1.22	0.02	0.690	1.61	0.03	0.862	2.01	0.03	1.72	3.97	0.07	58
59	0.085	0.237	0.003	0.169	0.430	0.006	0.339	0.816	0.013	0.508	1.20	0.02	0.678	1.59	0.03	0.847	1.97	0.03	1.69	3.90	0.06	59
60	0.083	0.234		0.167	0.428		0.333	0.803		0.500	1.18	0.02	0.667	1.56	0.03	0.833	1.94		1.67	3.84		60

NOTE. Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2. MEAN RATING TABLE FOR SMALL PRICE METER.

Based on Ratings of Nine Meters, Made During 1903, at Chevy Chase Lake, Maryland.

Equation Curve, { Between 0.0 and 3.9 Rev. per Sec., $v = 2.27609n + 0.01400$
" 3.9 " 7.0 " " $v = 2.44333n - 0.06242$

Time, in Seconds.	5 Revolutions.			10 Revolutions.			20 Revolutions.			30 Revolutions.			40 Revolutions.			50 Revolutions.			60 Revolutions.		
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.
30	0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381	
31	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014
32	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014
33	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011
34	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010
35	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009
36	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009
37	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009
38	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007
39	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008
40	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007

Time, in Seconds.	70 Revolutions.			80 Revolutions.			90 Revolutions.			100 Revolutions.			110 Revolutions.			120 Revolutions.			130 Revolutions.		
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.
30	0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381		0.167	0.381	
31	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014	0.161	0.367	0.014
32	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014	0.156	0.353	0.014
33	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011	0.152	0.342	0.011
34	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010	0.147	0.332	0.010
35	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009	0.143	0.323	0.009
36	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009	0.139	0.314	0.009
37	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009	0.135	0.305	0.009
38	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007	0.132	0.298	0.007
39	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008	0.128	0.290	0.008
40	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007	0.125	0.283	0.007

TABLE 2—*Continued.*

46	1.30	3.01		1.52	3.51		1.74	4.00		1.96	4.50		2.17	4.99		3.26	7.47		4.35	10.02		**46**
			0.05			0.07			0.08			0.09			0.11			0.16			0.23	
47	1.28	2.95		1.49	3.43		1.70	3.92		1.91	4.40		2.13	4.89		3.19	7.31		4.26	9.79		**47**
			0.06			0.07			0.08			0.09			0.10			0.15			0.22	
48	1.25	2.89		1.46	3.36		1.67	3.84		1.88	4.31		2.08	4.79		3.12	7.16		4.17	9.58		**48**
			0.06			0.07			0.08			0.09			0.10			0.15			0.21	
49	1.22	2.83		1.43	3.30		1.63	3.76		1.84	4.22		2.04	4.69		3.06	7.01		4.08	9.37		**49**
			0.06			0.07			0.07			0.08			0.09			0.14			0.20	
50	1.20	2.78		1.40	3.23		1.60	3.69		1.80	4.14		2.00	4.60		3.00	6.87		4.00	9.17		**50**
			0.05			0.06			0.07			0.08			0.09			0.13			0.19	
51	1.18	2.72		1.37	3.17		1.57	3.61		1.76	4.06		1.96	4.51		2.94	6.74		3.92	8.98		**51**
			0.05			0.06			0.07			0.08			0.09			0.13			0.18	
52	1.15	2.67		1.35	3.11		1.54	3.55		1.73	3.98		1.92	4.42		2.88	6.61		3.85	8.80		**52**
			0.05			0.06			0.07			0.07			0.08			0.12			0.17	
53	1.13	2.62		1.32	3.05		1.51	3.48		1.70	3.91		1.89	4.34		2.83	6.49		3.77	8.63		**53**
			0.05			0.06			0.06			0.07			0.08			0.12			0.16	
54	1.11	2.57		1.30	2.99		1.48	3.42		1.67	3.84		1.85	4.26		2.78	6.37		3.70	8.47		**54**
			0.05			0.05			0.06			0.07			0.08			0.11			0.15	
55	1.09	2.53		1.27	2.94		1.45	3.35		1.64	3.77		1.82	4.18		2.73	6.25		3.64	8.32		**55**
			0.04			0.05			0.06			0.07			0.07			0.11			0.15	
56	1.07	2.48		1.25	2.89		1.43	3.30		1.61	3.70		1.79	4.11		2.68	6.14		3.57	8.17		**56**
			0.04			0.05			0.06			0.06			0.07			0.11			0.14	
57	1.05	2.44		1.23	2.84		1.40	3.24		1.58	3.64		1.75	4.04		2.63	6.03		3.51	8.03		**57**
			0.04			0.05			0.06			0.06			0.07			0.10			0.14	
58	1.03	2.40		1.21	2.79		1.38	3.18		1.55	3.58		1.72	3.97		2.59	5.93		3.45	7.89		**58**
			0.04			0.05			0.05			0.06			0.07			0.10			0.13	
59	1.02	2.36		1.19	2.74		1.36	3.13		1.53	3.52		1.69	3.90		2.54	5.83		3.39	7.76		**59**
			0.04			0.05			0.05			0.06			0.06			0.10			0.13	
60	1.00	2.32		1.17	2.70		1.33	3.08		1.50	3.46		1.67	3.84		2.50	5.73		3.33	7.63		**60**

NOTE.—Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2—*Continued.*

Time, in Seconds.	60 Revolutions.			70 Revolutions.			80 Revolutions.			90 Revolutions.			100 Revolutions.			150 Revolutions.			200 Revolutions.			Time, in Seconds.
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	
30	2.00	4.60		2.33	5.35		2.67	6.11		3.00	6.87		3.33	7.63		5.00	11.61		6.67	15.69		30
31	1.94	4.45	0.15	2.26	5.18	0.17	2.58	5.92	0.20	2.90	6.65	0.22	3.23	7.39	0.24	4.84	11.22	0.39	6.45	15.16	0.53	31
32	1.88	4.31	0.14	2.19	5.02	0.16	2.50	5.73	0.18	2.81	6.45	0.21	3.12	7.16	0.23	4.69	10.85	0.37	6.25	14.67	0.49	32
33	1.82	4.18	0.13	2.12	4.87	0.15	2.42	5.56	0.17	2.73	6.25	0.19	3.03	6.94	0.22	4.55	10.50	0.35	6.06	14.20	0.46	33
34	1.76	4.06	0.12	2.06	4.73	0.14	2.35	5.40	0.16	2.65	6.07	0.18	2.94	6.74	0.20	4.41	10.18	0.33	5.88	13.77	0.44	34
35	1.71	3.95	0.11	2.00	4.60	0.13	2.29	5.25	0.14	2.57	5.90	0.17	2.86	6.55	0.18	4.29	9.87	0.31	5.71	13.36	0.41	35
36	1.67	3.84	0.11	1.94	4.47	0.12	2.22	5.10	0.14	2.50	5.73	0.16	2.78	6.37	0.17	4.17	9.58	0.29	5.56	12.97	0.39	36
37	1.62	3.73	0.10	1.89	4.35	0.11	2.16	4.97	0.13	2.43	5.58	0.15	2.70	6.20	0.16	4.05	9.30	0.28	5.41	12.60	0.37	37
38	1.58	3.64	0.10	1.84	4.24	0.11	2.11	4.84	0.12	2.37	5.43	0.14	2.63	6.03	0.15	3.95	9.04	0.26	5.26	12.26	0.35	38
39	1.54	3.55	0.09	1.79	4.13	0.10	2.05	4.71	0.12	2.31	5.30	0.13	2.56	5.88	0.15	3.85	8.80	0.24	5.13	11.93	0.33	39
40	1.50	3.46	0.09	1.75	4.03	0.10	2.00	4.60	0.11	2.25	5.16	0.12	2.50	5.73	0.14	3.75	8.58	0.22	5.00	11.61	0.31	40
41	1.46	3.37	0.08	1.71	3.93	0.09	1.95	4.48	0.11	2.20	5.04	0.12	2.44	5.60	0.13	3.66	8.37	0.21	4.88	11.32	0.30	41
42	1.43	3.30	0.08	1.67	3.84	0.09	1.90	4.38	0.10	2.14	4.92	0.11	2.38	5.46	0.13	3.57	8.17	0.20	4.76	11.08	0.28	42
43	1.40	3.22	0.08	1.63	3.75	0.09	1.86	4.28	0.10	2.09	4.81	0.11	2.33	5.34	0.12	3.49	7.98	0.19	4.65	10.76	0.27	43
44	1.36	3.15	0.07	1.59	3.66	0.08	1.82	4.18	0.09	2.05	4.70	0.10	2.27	5.22	0.11	3.41	7.80	0.18	4.55	10.50	0.26	44
45	1.33	3.08	0.07	1.56	3.58	0.08	1.78	4.09	0.09	2.00	4.60	0.10	2.22	5.10	0.11	3.33	7.63	0.17	4.44	10.26	0.24	45

TABLE 3.—Deviation of Velocity in Individual Current Meter Rating Tables from that of the Mean Table, Expressed as a Percentage of the Velocity of the Mean Table.

Revolutions per Second.	Velocity, Mean Table.	Meter Number.								
		738	739	788	789	790	792	793	794	795
	Feet per second.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per-cent.
0.1	0.269	−2.6	−1.5	−2.2	−0.74	−3.3	+7.4	+3.3	−3.0	+0.74
0.2	0.494	+0.40	−1.4	−0.20	−0.81	−1.8	+4.4	+1.8	−1.6	0.0
0.3	0.721	+1.1	−1.8	0.0	−1.5	−1.2	+3.0	+1.4	−1.1	0.0
0.4	0.945	+1.8	−1.6	+0.1	−1.4	−0.95	+2.3	+0.95	−0.95	−0.32
0.5	1.17	0.0	−1.7	0.0	−0.85	−0.85	+1.7	+0.85	−0.85	0.0
0.6	1.39	0.0	−1.4	+0.72	−0.72	0.0	+2.2	+1.4	−0.72	0.0
0.7	1.62	0.0	−1.2	0.0	−0.62	−0.62	+1.2	0.0	−0.62	0.0
0.8	1.84	0.0	−0.54	+1.1	−0.54	+0.54	+1.6	+0.54	0.0	−0.54
0.9	2.07	0.0	−0.97	+0.48	−0.48	0.0	+1.9	+0.97	0.0	0.0
1.0	2.30	−0.43	−0.87	+0.87	−0.43	0.0	+1.1	+1.7	+0.43	−0.87
1.5	3.44	−1.2	−0.58	+0.29	−0.87	−0.29	+0.87	+1.2	+0.29	−0.58
2.0	4.59	−0.87	−0.87	+0.22	−1.3	−0.44	+1.1	+1.1	+0.44	0.0
2.5	5.73	−0.35	−0.70	+0.35	−1.2	−0.35	+1.0	+1.2	+0.52	+1.7
3.0	6.87	−0.15	−1.0	0.0	−1.3	−0.15	+1.0	+1.2	+0.58	+0.44
3.5	8.01	+0.12	−1.0	+0.12	−1.4	+0.75	+0.62	+0.62	+0.37	+0.25
4.0	9.17	0.0	.....	+0.54	−0.11	−0.33	.....	.....	.....	.....
4.5	10.38	−0.48	.....	+0.29	+0.48	−0.10	.....	.....	.....	.....
5.0	11.58	−0.86	.....	+0.34	+0.69	−0.09	.....	.....	.....	.....
5.5	12.82	.....	.....	−0.23	+0.62	−0.31	.....	.....	.....	.....
6.0	14.06	.....	.....	−0.50	+0.71	−0.28	.....	.....	.....	.....
6.5	15.26	.....	.....	−0.52	+0.59	.....	.....	.....	.....	.....

Care of the Current Meter.

Although the current meter is substantially made, and will stand considerable hard usage, it needs special attention and care to insure its proper working.

In taking the meter apart, remove the tail vanes and the hanger stem; then loosen the set-screw to the contact chamber, and pull the chamber out by a slight twisting motion. Care must be taken to let the cups be free to turn, so that the worm gear on the upper end of the shaft can disengage itself from the teeth of the contact wheel. In handling the contact chamber, it is well to take off the cap, so that the gear-wheel can be seen during the operation. The cone point can then be taken out and the cups released by loosening the upper part of the shaft with a spanner wrench. This wrench is arranged so that all parts of the meter can be loosened with it.

In putting the meter together, first attach the cups to the cup shaft. In doing this, the upper part of the shaft should be inserted

through the upper hole of the yoke before it is screwed to the lower part. Care must be taken to place the cups so that they will move counter-clockwise. After the cups have been fastened to the shaft, insert the cone point and clamp it in place, and then insert the contact chamber. In replacing the contact chamber, the cups should be left free to move on the cone point, and care should be taken not to allow the cogs on the worm gear to catch on the teeth of the contact wheel. Before inserting the cone plug, the cone point should be adjusted and firmly secured with a lock-nut. The adjustment should allow a slight vertical motion of the cups.

There are two cases in which the meter can be carried: one is a box, 19 by 7 by 5 in., and the other is a box, 8½ by 6½ by 5 in. The larger box takes the meter with the tail attached, and allows sufficient room for carrying a small weight, a cable for suspending the meter, and miscellaneous tools. The small case is arranged with a shoulder-strap, and is just large enough to carry the meter and the tail when taken apart. The weights, the cable, and other equipment have to be carried in a separate case.

The following articles should accompany each current-meter equipment: A small screw-driver, a pair of parallel pliers with a wire cutter, a spanner wrench, an oil-can, a roll of adhesive tire tape, 25 ft. of metallic tape, a 50-ft. steel tape, an extra cone point, and extra set-screws.

In caring for and using a current meter, the points noted in the following paragraphs should be carefully considered:

1. Be sure that the set-screws are all tightened before putting the meter in the water; otherwise one of the parts may be lost.

2.—Loosen the sleeve-nut and see that the meter runs freely before beginning a measurement; and spin the meter cups occasionally during a measurement to see that they are running freely.

3.—See that the weights play freely on the stem, so as to take the direction of the current and thus avoid a drag on the line.

4.—If any apparent inconsistency in the results of an observation throws doubt on its accuracy, investigate the cause at once. Grass may be wound around the cup shaft; the cups may be tilted by tension on the contact wire; the channel may be obstructed immediately above the meter; the meter may be in a hole; or the cups may be bent so as to come in contact with the yoke.

5.—After a measurement, clean and oil the bearings (in order to prevent rust) and inspect the cone point.

6.—In packing the meter, turn the sleeve-nut to lift the cups from the cone point.

7.—Always see that the lock-nut on the cone point is firmly screwed against the cone plug.

8.—If the cone point is dulled, it can be sharpened with an oil-stone.

9.—In measuring low velocities, be sure that the meter is in a horizontal position. If it has a tendency to tip, it can be held in place by using a plug in the slot for the stem.

10.—Avoid taking measurements in velocities of less than $\frac{1}{2}$ ft. per sec., as the accuracy of the meter diminishes as zero velocity is approached. Compare velocities of 0.5 ft. per sec. and less, as shown in Table 3.

11.—For velocities of less than 1 ft. per sec., the bearing point should be sharp and smooth, as at the time of rating. As the velocity increases, the condition of the point is less important, for then the friction becomes a small factor.

12.—In taking measurements at high velocities, sufficient weight, or a stay-line, should be used to hold the hanger so that the meter will remain horizontal.

13.—In very shallow streams the meter should be suspended from the lower hole on the stem, and the weight should be placed above.

14.—If the cups of a small Price meter are bent, they may be easily put in shape by using a wood or metal bar with a round, smooth end.

15.—The telephone receiver is very sensitive to electric currents, and can be used to locate any break in the circuit. First try the telephone and battery together (Fig. 4) in a circuit having a make-and-break point, as at *a*. This may be done by using a knife blade or a screw-driver, making connection where the wires enter the plug. If there is no click in the 'phone, then the battery or the 'phone does not make a circuit. If there is a click, insert the meter in the line and test for a contact in the meter head (Fig. 5) by revolving the meter wheel. If the meter is all right, put the meter cord in the circuit and test both sides by making double connection and touching alternate sides of the line, *a* (Fig. 6).

16.—When the meter is not in use, disconnect the meter line from the battery, so that it will not become exhausted.

17.—When a measurement has been completed, the solution may be left in the wet cell if the zinc pole and stopper are replaced by a rubber cork.

18.—Never let the bisulphate dry in the cell, as it forms a hard cake and polarizes the battery.

19.—Do not let any bisulphate of mercury remain loose in the meter box; if it gets into the meter bearings it will corrode them.

20.—The zinc pole in the bisulphate cell sometimes gets pushed down so that it touches the bottom of the cell, in which case the cell is short-circuited and the recorder will not indicate. To test this, lift the plug a little way out of the battery and see if there is a flow of current.

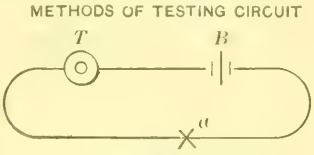


FIG. 4.

21.—Keep the points clean where the battery makes contact with the metal plates.

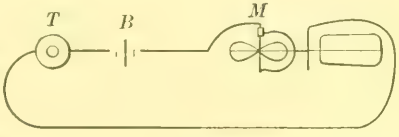


FIG. 5.

22.—The amount of current necessary to work with a telephone receiver is very small, and a battery may be serviceable even though nearly exhausted.

23.—If care is taken, it is very improbable that the telephone receiver will get out of order.

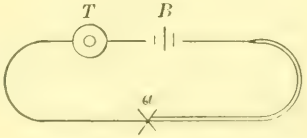


FIG. 6.

24.—Do not strike the telephone receiver, as a heavy jar will to a greater or less extent demagnetize the pole pieces, and to that extent will injure the receiver.

25.—Care must be taken not to short-circuit the dry battery when the meter is not in use, as in that way the cell becomes exhausted in a short time, the energy being used in heating the cell. To avoid this, the poles are wound with adhesive tape.

26.—If a dry cell which has been long in stock fails to work well, punch two nail holes in the wax on top of the cell and put it in water

over night, when it will absorb enough moisture to renew it. The holes should then be coated over by heating the wax with a match and pressing it into place, or by pouring in melted paraffin. A cell which has been exhausted by use is not benefited much by this treatment. The life of a cell depends largely on the amount of leakage in the line during use.

Discharge Measurements with the Current Meter.*

The measurement of stream discharge, for which the current meter was devised, consists in the determination of the quantity of water flowing past a given section of a stream in a given time. This quantity is obtained as the product of two factors: (*a*) The area of the cross-section of the stream, which is determined by sounding; and (*b*) the velocity of the water, which is measured with the current meter. The usual unit for expressing the discharge of a stream is the second-foot—an abbreviation for cubic foot per second—equivalent to the quantity of water flowing in a stream 1 ft. wide and 1 ft. deep, at a velocity of 1 ft. per sec.

Sections at which discharge measurements are regularly made are known as gauging stations, and may be divided into four classes, depending on whether the observations of velocity and depth are made from (*a*) a bridge, (*b*) a cable, (*c*) a boat, or (*d*) by wading (Plates XXVII and XXVIII). The method of making measurements is the same for all stations.

The ideal measuring section should be perpendicular to the thread of the velocity of the water and at a point on the stream where the conditions of bank and bed, both above and below the section, are permanent. This section, Fig. 7, is divided into partial areas bounded by perpendiculars terminating at points in the surface. These are known as measuring points, as the observations of the depth of the stream and the mean velocity of the water in the vertical are made at such points. The distance between the measuring points will depend on the size and character of the stream. The points should be spaced so as to show any irregularities either in the cross-section or in the velocity. If the section is to be used for future measurements, the measuring points should be indicated by permanent marks placed

* A detailed description of the collection and use of stream-flow data and the preparation of estimates of discharge may be found in "River Discharge," by Hoyt and Grover, published by Wiley & Sons, New York.

FIG. 1.—MEASUREMENT BY WADING, GRAND CENTRAL RIVER, ALASKA.

FIG. 2.—MEASUREMENT AT CABLE GAUGING STATION, YAKIMA RIVER, NEAR UNION GAP, WASH.

at intervals of 5 or 10 ft. on the bridge rail or floor of a bridge station, on the cable of a cable station, or on the tape or tagged line usually stretched across a stream when the measurements are made from a boat or by wading.

The procedure in the measurements varies somewhat with the sounding appliance. If the soundings are made with meter and cord, the depth and velocity are usually observed at each successive measuring point across the stream. If a rod or sounding line is used without the meter, soundings are made at all the measuring points before taking the velocities.

TYPICAL CROSS-SECTION OF STREAM
TO ILLUSTRATE DISCHARGE MEASUREMENT

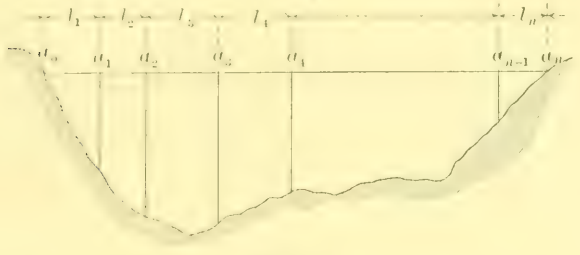


FIG. 7.

In sounding with the rod, care is taken that it does not sink into the bed of the stream, and that the reading is not too high on account of the water running up on it. The soundings from bridges or cables are usually made with the weight and line, and in such cases care is taken to eliminate the error due to the weight being carried down the stream or to the bowing of the line. Soundings with the line are most readily taken as follows: The weight and line are lowered until the weight rests on the bed of the river directly under the measuring point, and, with the line taut, a point is marked on it opposite a fixed point on the bridge or car; the weight is then raised until it just touches the surface of the water and the length of the sounding line that passes the fixed point is measured. The depth is measured by placing the end of a linen or metallic tape opposite the fixed starting point on the sounding line, grasping both the line and the tape in the hands, and drawing up the line and tape without permitting them to slip on each other until the weight reaches the surface of the water. The length of line thus drawn up, representing the depth of the water, is then

read directly from the tape. This measurement can usually be made by one person, even when the water is from 10 to 12 ft. deep.

In making a velocity measurement, the meter is held at the point in the stream at which it is desired to ascertain the velocity of the current. The wheel is allowed to revolve for a few seconds, in order that it may adjust itself to the current, after which the time for a given number of revolutions is noted, from which the velocity is obtained from the rating table for the meter. The time should be about 40 sec., and the number of revolutions will depend on the velocity of the water. A check on the work is made by repeating the observation. If the run is not repeated, a check can be obtained by noting mentally the time for each five revolutions. A stop-watch is used for observing the time, and the record is made to the nearest 0.2 sec., usually the smallest interval indicated with the ordinary stop-watch.

In discharge measurements, the mean horizontal velocity in a vertical at the measuring points is desired. Various methods are used for this determination, among which the following four are most common: (*a*) Vertical velocity-curve method, (*b*) two-tenths-and-eight-tenths-depth method, (*c*) six-tenths-depth method, and (*d*) subsurface method.

(*a*).—By the vertical velocity-curve method, measurements of horizontal velocity are usually made just beneath the surface, at 0.5 ft. below the surface, and at each fifth to each tenth of the depth from the surface to the bed of the stream, and as near the bottom as possible. These measured velocities, when plotted with depths as ordinates and velocities as abscissas, define for each vertical the vertical velocity-curve, which shows the velocity at every point in the vertical, and from which its mean velocity can be determined by dividing the area bounded by the curve and the axis of the ordinates by the mean depth. The following three methods are used in this determination:

1.—Determine the area with the planimeter and divide by the depth.

2.—Divide the area into any number of sections of equal depth, usually ten, and take the mean of the velocities at mid-point of each of these sections as the mean velocity.

3.—Divide the area into sections of convenient depth, which will be equal except for the bottom section, which may have an odd depth.

FIG. 1.—MEASUREMENT AT BRIDGE GAUGING STATION.

FIG. 2.—MEASUREMENT AT BOAT GAUGING STATION.

If the bottom section is the same depth as the others, take the mean of the middle ordinates of each section for the mean velocity. If the bottom section is an odd depth, multiply its mean velocity by the ratio of its depth to that of the common depth of the other sections, and add this product to the sum of the middle ordinates of the other sections, and divide by the number of sections.

Studies of vertical velocity-curves taken on many streams under various conditions of depth, velocity, and roughness of bed, show that these vertical velocity-curves have approximately the form of a parabola in which the axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water. From the maximum the velocity decreases gradually upward to the surface and downward nearly to the bottom, where it changes more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

The vertical velocity-curve method is valuable as a basis for the comparison of all other methods, for determining the coefficients to be used in reducing the values obtained by other methods to the true value, for use under new and unusual conditions of flow, and for measurements under ice.

(*b*).—In the two-tenths-eight-tenths method, the observations of velocity are taken at two points located at depths from the surface of 0.2 and 0.8 of the depth at the measuring point. The mean velocity is taken as the mean of the velocities at these two points. This method is based on the theory that the vertical velocity-curve is a parabola, as already stated, in which case the mean of the ordinates at 0.2114 and 0.7886 gives the mean ordinate. A study of a large number of vertical velocity-curves shows that this holds true in Nature; and experience proves that this method gives more consistent results than any of the others except the vertical velocity-curve method.

(*c*).—In the six-tenths method, the observation of the velocity is taken at a depth from the surface equal to 0.6 of the depth of the stream. This method is also based on the theory that the vertical velocity-curve is a parabola with the maximum abscissa between zero and one-third the depth, in which case the mean ordinate is between 0.58 and 0.67 of the depth from the surface. A study of a large number of vertical velocity-curve measurements shows that the mean depth

of the mean velocity is approximately 0.6 of the depth. This method has the advantage of ease in making the measurement, and gives very satisfactory results but not as good as those obtained by the two-tenths-eight-tenths method.

(*d*).—In the sub-surface method, the measurement of velocity is made at from 0.5 to 1 ft. below the surface, depending on the depth of the stream. The meter is held at sufficient depth to be out of any surface disturbance. When this method is used, the velocity must be reduced by a coefficient to obtain the mean velocity. This coefficient varies between 78 and 98%, depending on the depth and the velocity of the stream. The deeper the stream and the greater the velocity, the greater the coefficient. For average streams in moderate freshets, a coefficient of 90% should be used; in flood work, a coefficient of from 90 to 95%; and for streams at ordinary stages, from 85 to 90 per cent.

The results of a large number of vertical velocity-curve measurements, giving a comparison of the two-tenths-eight-tenths method, the six-tenths method, and the sub-surface method are given in Table 4.

Independent discharge measurements, as a rule, are of but little value in hydraulic work unless they are taken at stages which are known to be either extremely low or extremely high. In ordinary work it is necessary to make a series of measurements which, with daily gauge heights of the flow of the stream, enable the computation of the total flow of the stream and also its distribution. In connection with the individual measurements, therefore, it is necessary to observe gauge heights and take full notes of the conditions under which the measurement is made, in order to enable the construction of a station rating curve and estimate the daily discharge.

Computations of Current-Meter Discharge Measurements.

The computations of current-meter discharge measurements are usually made to determine: (*a*) the total area, (*b*) the mean velocity, and (*c*) the total discharge of the stream. The observed data from which these computations are made consist of (*a*) soundings at known intervals across the stream, (*b*) the mean velocity in the vertical at each sounding point, and (*c*) the distance between the points of measurement (Table 5).

The mean velocity, area, and discharge are computed independently

for each partial area included between perpendiculars from successive measuring points. The total discharge is the sum of the partial discharges. The computation of the partial discharge eliminates errors which would arise from the distribution of conditions existing in one part of the channel to parts to which they do not apply. The mean velocity is determined by dividing the total discharge by the total area of the cross-section.

The formulas used in connection with computations, may, in general, be classed as rectilinear and curvilinear depending on the assumption that the bed of the stream and the horizontal velocity-curve vary from the measuring point either as a straight line or as a series of curved lines. A comparison of the computation of discharge measurements by various formulas, has been prepared by J. C. Stevens, Assoc. M. Am. Soc. C. E., District Engineer of the United States Geological Survey.*

In this discussion it is shown that the rectilinear formula:

$$D = l_1\left(\frac{d_0+d_1}{2}\right)\left(\frac{v_0+v_1}{2}\right)+l_2\left(\frac{d_1+d_2}{2}\right)\left(\frac{v_1+v_2}{2}\right)+$$
$$\ldots\ldots\ldots\ldots\ldots\ldots l_n\left(\frac{d_{n-1}+d_n}{2}\right)\left(\frac{v_{n-1}+v_n}{2}\right)$$

gives the most accurate results and is readily used.

In this formula, d_0, d_1, d_2.....d_n and v_0, v_1, v_2.....v_n are the depths and velocities at the respective measuring points, a_0, a_1, a_2.....a_n, which are spaced at the distances, l_1, l_2, l_3.....l_n (Fig. 7). The area between any two perpendiculars at measuring points is equal to the mean of the depths at such points multiplied by the distance. Similarly, the mean velocities for the area between the two perpendiculars is equal to the mean of the velocities at the two perpendiculars. The product of this area and the mean velocity is equal to the discharge for the partial area between the two perpendiculars. Summing these partial areas and discharges gives the total area and total discharge. The mean velocity for the section is equal to the total discharge divided by the total area. Table 5 gives the field notes and computations for a typical current-meter measurement.

Low-Water Measurements.

At many stations the velocity of the river at low stages is so small that it may be advisable to find a section near by in which

*Engineering News, June 25th, 1908.

Measurements Prior to 1905.										
Susquehanna	McCall Ferry, Pa	73	3.3–30.0	1.21–5.80	68		0.94		0.92	1.07
Susquehanna	McCall Ferry, Pa	68	5.0–36.0	1.40–9.70	73		0.97		0.90	
Susquehanna	Harrisburg, Pa	20	3.2– 8.0	1.52–2.71	61		1.01		0.85	1.08
Susquehanna	Binghamton, N. Y	36	2.5– 8.1	0.80–4.86	61		0.99			0.96
Chenango	Binghamton, N. Y	34	1.7– 7.4	0.46–3.38	62		0.98			1.04
Catskill Creek	South Cairo, N. Y	47	2.1– 7.0	0.27–3.91	58		1.02		0.83	
Delaware (E. Br.)	Hancock, N. Y	6	2.1– 4.8	0.75–4.12	59		1.02		0.82	
Delaware (W. Br.)	Hancock, N. Y	12	1.7– 4.2	1.67–4.45	58		1.02		0.83	
Esopus Creek	Kingston, N. Y	53	1.9–14.0	0.65–5.04	62		1.00		0.88	
Esopus Creek	Olivebridge, N. Y	68	0.9– 5.2	0.42–7.62	58		1.01		0.69	
Fishkill Creek	Glenham, N. Y	22	2.1– 5.0	0.26–1.75	59		1.04		0.79	
Housatonic	Gaylordsville, Conn	36	2.1– 6.2	0.67–3.66	58		1.01		0.83	
Rondout Creek	Rosendale, N. Y	21	4.4– 8.2	0.35–2.55	61		0.99		0.84	
Schoharie Creek	Prattsville, N. Y	11	1.1– 6.6	0.52–3.20	58		1.01		0.80	
Tenmile	Dover Plains, N. Y	33	1.8– 7.3	0.24–2.15	61		0.99		0.88	
Wallkill	New Paltz, N. Y	52	3.3–17.3	0.18–4.72	54		0.98		0.85	
Wappinger Creek	Wappinger Falls, N. Y	17	2.5– 4.7	0.58–2.11	58		1.03		0.78	
Appomattox	Mattoax, Va	8	2.5– 4.9	0.92–2.30	62		1.00		0.81	1.11
James	Cartersville, Va	6	3.0– 4.4	1.88–2.37	60		1.01		0.88	1.08
Roanoke	Roanoke, Va	8	1.8– 3.3	0.87–1.96	61		0.99		0.81	1.01
Staunton	Randolph, Va	20	2.0– 6.0	1.35–3.11	66		0.96		0.95	1.12
Dan	Madison, N. C	42	1.3– 3.8	0.73–3.00	62		0.99		0.86	1.01
Dan	South Boston, Va	13	3.5– 9.0	0.90–2.41	64		0.98		0.90	1.09
Reddie	North Wilkesboro, N. C	3	1.7– 2.5	1.62–2.01	58		1.02		0.83	0.97
Yadkin	North Wilkesboro, N. C	33	2.7– 6.4	1.20–3.16	66		0.96		0.91	1.12
Yadkin	Salisbury, N. C	10	3.1–11.3	0.96–3.23	59		1.02		0.81	1.06
Catawba	Morgantown, N. C	39	1.9– 5.0	0.74–2.84	61		1.00		0.82	1.04
Catawba	Mount Holly, N. C	2	6.3– 7.0	0.49–0.74	58		1.00		0.88	1.08
Catawba	Belmont, N. C	1	3.5	1.54	59		1.01		0.80	1.05
Wateree	Camden, S. C	4	12.3–17.7	1.33–1.83	58		1.01		0.90	1.13
Broad	Alsten, S. C	4	5.0– 8.9	2.79–4.00	65		0.96		0.89	1.22
Saluda	Waterloo, S. C	3	3.0– 8.5	1.76–2.01	62		0.97		0.82	1.03
Watauga	Elizabethton, Tenn	10	2.6– 5.3	0.34–1.01	61		0.98		0.82	1.07
South Fork Holston	Bluff City, Tenn	13	1.9– 4.1	1.08–6.77	62		1.02		0.80	1.05
Nolichucky	Greeneville, Tenn	22	1.6– 5.1	0.49–2.69	59		1.02		0.80	1.02
French Broad	Oldtown, Tenn	9	4.2– 8.6	0.53–2.18	62		0.99		0.86	1.07
Pigeon	Newport, Tenn	4	4.4–11.6	0.82–1.14	61		0.99		0.91	1.15
Tuckaseegee	Bryson City, N. C	36	3.5– 8.5	0.50–3.39	64		0.97		0.93	1.09
Little Tennessee	Judson, N. C	11	2.7– 6.0	3.84 7.45	59		1.03		0.83	1.06
Mean of 910 curves					61		1.00		0.85	1.07
Highest			36.0	9.70	73		1.04		0.95	1.22
Lowest			1.1	0.18	58		0.94		0.78	0.96

* From "River Discharge."

TABLE 4.—SUMMARY OF VERTICAL VELOCITY CURVES IN OPEN CHANNELS.*

Stream and locality.		Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.				
							Six-tenths depth.	Mid-depth.	Top.	Mean of top and bottom. $\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
MEASUREMENTS SINCE 1905.											
Ohio	Wheeling, W. Va.	14	5.3–20.8	0.92–4.43	66	13	0.98	.....	0.93	.....	1.005	.....
Monongahela	Lock No. 4, Pa.	7	10.9–17.5	0.74–1.13	68	0	0.96	.....	0.85	.....	0.990	.....
St. Mary	Cardston, Alberta	8	2.7– 3.9	3.78–6.32	58	.....	1.01	.....	0.88	.....	1.001	.....
Jefferson	Sappington, Mont.	7	2.7– 5.7	1.78–3.45	63	.....	0.97	.....	0.84	.....	0.997	.....
Gallatin	Logan, Mont.	7	3.7– 5.5	0.88–3.85	62	.....	0.98	.....	0.85	.....	0.991	.....
Missouri	Cascade, Mont.	32	2.1–12.0	0.79–3.04	62	.....	0.99	.....	0.88	.....	1.002	.....
Marias	Shelby, Mont.	20	1.9– 4.2	0.57–4.52	60	.....	1.01	.....	0.86	.....	1.004	.....
Yellowstone	Glendive, Mont.	11	4.3– 8.5	2.42–3.79	60	.....	1.00	.....	0.89	.....	1.003	.....
Bighorn	Fort Custer, Mont.	8	3.9– 7.4	1.22–2.60	59	.....	1.00	.....	0.86	.....	0.998	.....
Shoshone	Cody, Wyo.	16	4.3– 7.3	4.93–8.45	58	13	1.02	0.96	0.87	.....	1.00	.....
North Platte	Saratoga, Wyo.	8	3.9– 5.1	1.82–3.61	63	26	0.98	.....	0.94	1.31	1.026	1.09
Pecos	Dayton, N. Mex.	10	4.1– 8.2	1.68–2.81	58	.....	1.03	.....	0.82	.....	1.003	.....
Pecos	Pecos, Tex.	6	2.2– 6.0	1.52–3.06	63	.....	0.97	.....	0.98	.....	1.004	.....
Green	Greenriver, Wyo.	36	2.2– 6.4	1.05–4.12	58	14	1.01	.....	0.90	1.20	1.026	1.06
Yampa	Steamboat Springs, Colo.	8	4.2– 5.2	7.13–8.42	59	7	1.01	.....	0.83	1.10	1.003	1.02
Grand	Hot Sulphur Springs, Colo.	6	2.8– 8.2	0.89–2.87	63	23	0.98	.....	0.85	1.30	1.01	1.03
Grand	Kremmling, Colo.	15	9.2–27.5	1.11–4.02	65	24	0.97	.....	0.93	1.16	1.001	1.03
Gunnison	Cimarron, Colo.	9	4.3–15.5	3.83–8.70	64	19	0.99	.....	0.92	1.06	1.005	1.02
Gunnison	Cory, Colo.	5	4.9–10.4	1.63–9.59	63	17	0.99	.....	0.92	1.09	1.007	1.01
Big Blackfoot	Bonner, Mont.	7	4.2– 6.8	1.41–2.49	62	.....	0.98	.....	0.85	.....	1.016	.....
Clark Fork	Missoula, Mont.	5	5.4– 7.6	1.72–2.41	60	10	1.00	.....	0.79	.....	0.997	.....
Yakima	North Yakima, Wash.	7	4.9– 7.6	1.58–2.56	59	.....	1.01	.....	0.83	1.06	0.999	1.00
Yakima	Yakima, Wash.	50	3.2–12.6	0.65–8.56	59	.....	1.01	.....	0.85	1.04	1.003	0.99
Yakima	Kiona, Wash.	20	2.0– 6.1	0.25–3.24	61	.....	0.98	.....	0.84	1.05	1.00	0.98
Kachess	Easton, Wash.	5	3.4– 4.3	1.44–4.78	61	.....	0.99	.....	0.85	1.04	1.009	0.99
Cle Elum	Roslyn, Wash.	20	4.7–11.7	0.37–3.89	58	.....	1.01	.....	0.84	1.09	0.995	1.02
Naches	North Yakima, Wash.	25	2.4– 6.0	0.65–7.38	58	.....	1.01	.....	0.85	1.04	0.998	0.99
Snake	Minidoka, Idaho	14	4.0–10.7	1.13–3.05	62	20	0.99	.....	0.89	.....	0.994	.....
Big Lost	Mackay, Idaho	7	2.4– 3.7	1.23–3.85	71	42	0.95	.....	0.95	.....	0.993	.....
Palouse	Hooper, Wash.	28	1.6– 8.0	0.97–6.49	60	.....	1.00	.....	0.84	1.06	1.004	1.00
John Day	McDonald, Ore.	15	3.6– 5.9	1.87–4.44	65	.....	0.98	.....	0.89	.....	.....	.....
Umatilla	Pendleton, Ore.	13	2.2– 7.5	0.47–2.73	64	.....	0.97	0.90	0.87	.....	0.970	.....
Deschutes	Lava, Ore.	10	3.0– 4.1	0.88–1.98	66	.....	0.99	0.94	0.92	.....	0.994	.....
West Fork Deschutes	Lava, Ore.	27	1.7– 7.1	0.58–2.65	64	.....	0.98	0.96	0.89	.....	0.992	.....
Mean of 476 curves					62	.....	0.99	.....	0.87	.....	1.001	.....
Highest			27.5	9.59	71	.....	1.03	.....	0.98	.....	1.026	.....
Lowest			1.6	0.25	58	.....	0.95	.....	0.79	.....	0.970	.....

TABLE 4.—*Continued.*

Stream and locality.	Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.					
						Six-tenths depth.	Mid-Depth.	Top.	Mean of top and bottom.	$\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
Shallow Streams with Rough Beds.											
St. Mary.....Dam Site, Mont.....	24	1.8- 4.2	2.69-6.36	60	9	1.00	0.93	0.86	.....	1.01	.....
Swiftcurrent Creek....Babb, Mont.....	16	1.6- 3.2	3.28-6.81	57	0	1.03	0.96	0.87	.....	1.004	.....
South Fork Shoshone.Marquette, Wyo.....	20	1.8- 3.6	3.61-6.79	58	0	1.02	0.93	0.82	.....	1.016	.....
Ana.....Summer Lake, Ore.....	6	1.8- 2.4	1.83-3.37	63	10	0.97	0.92	0.89	.....	1.00	.....
Tieton.....North Yakima, Wash.....	14	1.8- 4.5	4.42-8.04	56	0	1.04	0.96	0.90	.....	1.00	0.98
Wallowa.....Wallowa, Ore.....	4	2.2- 2.9	4.16-4.93	58	0	1.04	0.94	0.78	.....	1.004	.....
Kruzgamepa.....Salmon Lake, Alaska.....	108	1.6- 3.6	1.43-4.70	64	33	0.97	0.91	0.86	.....	1.004	0.99
Small streams.....Seward Peninsula, Alaska...	27	1.7- 2.5	2.15-3.47	61	14	1.00	0.93	0.84	.....	1.004	1.00
Mean of 219 curves.....	.....	.....	.....	60	7	1.01	0.92	0.84	.....	1.005	0.99
Highest.....	.....	4.5	8.04	64	33	1.04	0.96	0.89	.....	1.016	.....
Lowest.....	.....	1.6	1.43	56	0	0.97	0.91	0.78	.....	1.00	.....

TABLE 5.—Current-Meter Discharge Measurement Notes.

Date, Dec. 12, 1908, 9.15 A. M. Stream, Rock Cr. Party, F. F. Henshaw. Locality, Mass. Ave. Bridge, D. C. Meter No. 112. Gauge height, beg. 1.97, end 2.03, mean 2.00. Total area, 67. Mean velocity, 2.15. Discharge, 144.

Observations.							Computations.				
					Velocity.						
Distance from initial point.	Depth.	Depth of observation	Time, in seconds.	Revolutions.	At point.	Mean in vertical.	Mean in section.	Mean depth.	Width.	Area.	Discharge.
0	0.0	....	.....	0	.....	0.00	.....	.....	.....	.....	.....
							0.48	0.2	2	0.4	0.2
2	0.4	0.21	46	20	.....	0.96					
							1.22	0.5	2	1.0	1.2
4	0.6	0.36	44.4	30	.....	1.49					
							1.60	0.75	2	1.5	2.4
6	0.9	0.54	38.6	30	.....	1.71					
		0.23	42.4	50	2.63		1.88	1.0	4	4.0	7.5
10	1.1	0.92	44	30	1.50	2.06					
		0.3	31.4	40	2.84		2.16	1.3	2	2.6	5.6
12	1.5	1.2	39.2	30	1.68	2.26					
		0.36	39.8	50	2.79		2.24	1.65	2	3.3	7.4
14	1.8	1.44	39.6	30	1.67	2.23					
16	1.9	.....	.....	.....	.....	.....	2.28	1.9	4	7.6	17.3
		0.38	38.8	50	2.87						
18	1.9	1.52	36.8	30	1.80	2.34					
		0.4	36.6	50	3.05		2.42	1.95	4	7.8	18.9
22	2.0	1.6	34.2	30	1.94	2.50					
		0.38	38	50	2.93		2.49	1.95	4	7.8	19.4
26	1.9	1.52	43.8	40	2.02	2.48					
		0.36	39		2.85		2.57	1.85	4	7.4	19.0
30	1.8	1.44	45	50	2.47	2.66					
		0.32	45	50	2.47		2.35	1.7	4	6.8	16.0
34	1.6	1.28	40.8	30	1.62	2.04					
		0.26	32		2.08		1.95	1.45	4	5.8	11.3
38	1.3	1.04	40.4	30	1.63	1.86					
		0.24	49.4	50	2.25		1.90	1.25	2	2.5	4.8
40	1.2	0.96	40	30	1.65	1.95					
							1.88	1.15	2	2.3	4.3
42	1.1	0.66	36.4	30	.....	1.82					
							1.65	1.0	3	3.0	5.0
45	0.9	0.57	44.8	30	.....	1.48					
							1.36	0.9	3	2.7	3.7
48	0.9	0.54	35.2	20	.....	1.25					
							0.88	0.45	1	0.4	0.4
49	0.0	.....	.....	.....	.....	0.50					
									49	66.9	144.4

the meter measurement may be made by wading. In order to obtain a suitable section, it may be necessary to cut off part of the flow by damming the stream and modifying the channel so as to get sufficient depth and velocity for measuring.

For such measurements, the meter clamped on the rod (Plate XXIII) is the most satisfactory; in this the Price acoustic attachment is especially useful. In making a wading measurement, a tape line is

stretched across the stream in order to mark the measuring points. The engineer stands below this tape line and to one side of the meter (Fig. 1, Plate XXVII) in order that he may not disturb the action of the meter. Iron rods, $\frac{3}{8}$ in. in diameter and 3 or 4 ft. long, and having a split at the top, may be used conveniently to support the tape when making such measurements.

Measurements Under Ice.

Measurements under ice should always be made by the vertical velocity-curve method, or by the two-tenths-eight-tenths method, the latter, by many comparisons with vertical velocity-curves, having been shown to give results closely approximating the mean velocity. In work of this class, if the water is not too deep and the velocity too great, the meter may best be held on the rod.

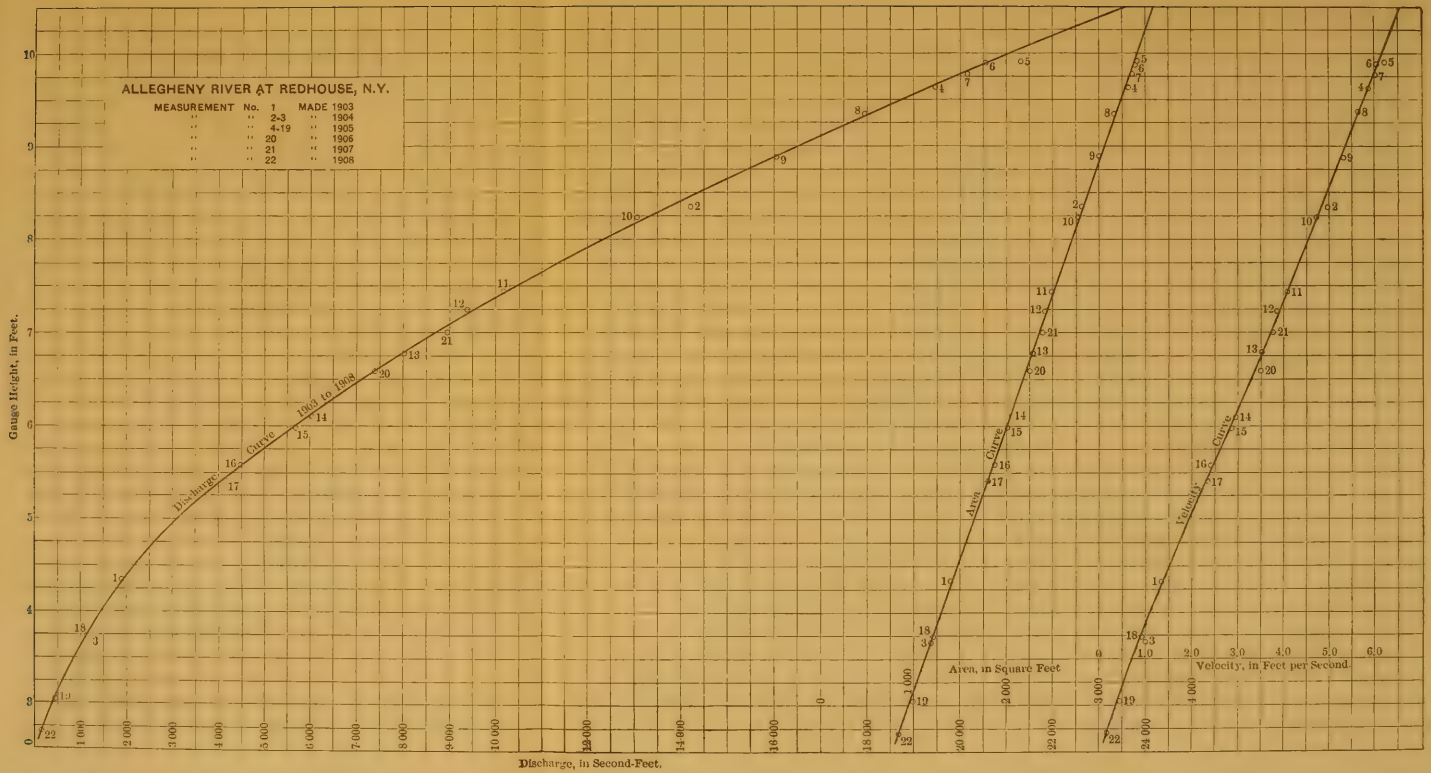
Measurements in Artificial Channels.

Current-meter measurement of the flow of water in artificial channels must be made with special care, as the laws of flow for open channels are not always applicable to artificial ones, because the water level may be subjected to disturbing influences such as undercurrents caused by intakes and outlets at rapid velocities.

Accuracy and Reliability of the Current Meter.

No attempt will be made herein to review the numerous discussions on the accuracy and use of the current meter. It is unfortunate that, in these discussions, most of the adverse arguments have been based on personal opinions rather than on the results of extended meter observations, and that from them there has resulted somewhat of a prejudice against the meter by those who are not familiar with its use and limitations. This prejudice, however, is rapidly disappearing, and, with increased use, the meter is coming to be recognized as a standard instrument in hydraulic investigations.

In the consideration of the accuracy of the results obtainable with the current meter, account should be taken of the use to be made of the data. It must be remembered that both the total flow of the stream and its distribution over the drainage area are constantly changing, and that the conditions existing at any time may probably never occur again. The flow which may be expected in any given stream, therefore, can be determined only by studying a series of


ALLEGHENY RIVER AT REDHOUSE, N.Y.
MEASUREMENT No. 1 MADE 1903
" " 2-3 " 1904
" " 4-19 " 1905
" " 20 " 1906
" " 21 " 1907
" " 22 " 1908
Discharge Curve 1903 to 1908
Area Curve
Velocity Curve
Gauge Height, in Feet.
Discharge, in Second-Feet.
Area, in Square Feet
Velocity, in Feet per Second.

records extending over a long period; for this reason ultra-refined measurements would have no more practical value than records having a reasonable degree of accuracy.

As with most instruments, the accuracy and reliability of the current meter depend largely on the care taken in the measurement and the method used. In general, the errors which have been attributed to the meter have been due wholly to the fact that it was operated by men who were unfamiliar with it and with the conditions under which it could be used.

Its reliability is best shown by the results of the measurements taken at gauging stations on streams with permanent beds. These results show that measurements taken with different meters and by different operators at intervals throughout a series of years plot on a well-defined curve, Plate XXIX.

An examination of the rating tables (Table 3) for various meters indicates that, barring a very slight variation due to slight differences in manufacture, a standard table could be used for all meters and still bring the results within the limit of error which should be expected in this work. The re-ratings of the meters, as used in the field, also indicate that they are not subject to great change, if they have not been damaged and if the pivots are kept in good condition. It is also noted that the meter is most reliable for velocities greater than 1 ft. per sec., which is the most common velocity in stream gauging.

A review of all the available information in regard to the accuracy of the meter, and of the data collected by the engineers of the United States Geological Survey during the last two decades, shows that the current meter is wholly reliable for general stream-gauging work, and that, in general, the errors due to the instrument when properly used will not exceed 5 per cent. It is believed that the results of the greater number of measurements are in error less than that amount.

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PAPERS AND DISCUSSIONS

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COMPUTATION OF STRESSES IN OPEN-WEBBED ARCHES WITHOUT HINGES.*

By C. W. Hudson, M. Am. Soc. C. E.

The principles underlying the following method are often used in finding the stresses in certain classes of statically indeterminate structures built of elastic material, but, as far as the writer is aware, they have never been applied to finding the stresses in an open-webbed arch rib without hinges.

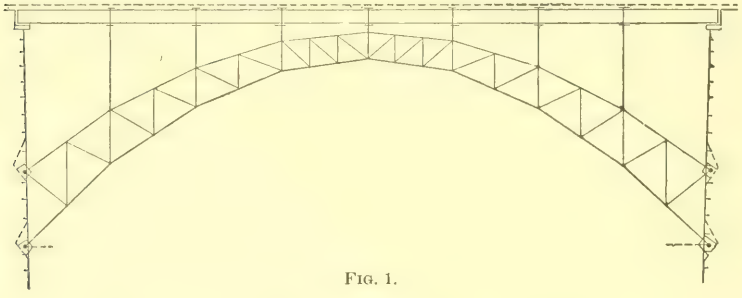

Fig. 1.

If such an arch rib be considered cut, at the crown, by a vertical plane, it is divided into two parts which are each statically determinate.

For the purpose of illustration, assume an arch rib as shown in Fig. 1. Consider the rib cut at the center by a vertical plane; this divides the rib into two parts as shown in Figs. 2 and 3, the vertical

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

member at the center being considered divided into two equal parts, one of which is taken with Fig. 2 and the other with Fig. 3.

It is perfectly clear that a load at any point on the left half produces stresses in the truss members of that half which may be determined by statics when each half acts as an independent cantilever truss, and that no stresses can be produced in the right half by this load when the two halves are not connected.

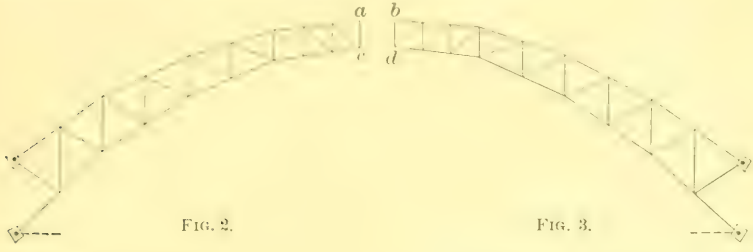

FIG. 2. FIG. 3.

If we suppose the two halves to be connected by joining the points, *a* and *b*, then, under any condition of loading, these two points have the same motion, as they are the same point. Therefore, for a load on the left half it is clear that the stress in any member of that half is that due to the given load when the half is considered as a separate structure modified by the stress acting between *a* and *b* caused by connecting these points, and the stress in any member of the right half is that due to the stress acting between *a* and *b* caused by connecting these points. No matter what the nature and direction of the force produced by joining *a* and *b*, it may be considered replaced by a horizontal and a vertical component.

If we suppose the two halves to be further connected by joining the points, *c* and *d*, then *a* and *b* have the same motion, and *c* and *d* have the same motion under any condition of loading. Therefore, for a load on the left half and the right and left halves connected at *a* and *b* and at *c* and *d*, the stress in any member of the left half is that due to the load on that half, when the half acts as a simple cantilever modified by the forces produced at *a* and *c* by connecting *a* to *b* and *c* to *d*, and the stress produced in any member of the right half is that due to the forces at *b* and *d* caused by connecting *a* to *b* and *c* to *d*. It hardly need be mentioned that the forces, due to connecting *a* to *b* and *c* to *d*, at *a* and *c*, are equal and opposite to those at *b* and *d*, respectively.

No matter what the amount and direction of the forces acting between a and b, and c and d, they may each be replaced by their horizontal and vertical components. For the two halves connected and subjected to any loading, the forces produced by thus joining them may be represented by the four forces of Fig. 4, H_a, V_a, H_c, and V_c, the direction of which may be as shown or the opposite and the magnitude of which is to be determined. Knowing that for elastic structures deflections are proportional to the loads that produce them, four equations between the unknown forces, H_a, V_a, H_c, and V_c, and certain easily determined deflections for the half arch acting independently, may readily be written. The solution of the four simultaneous equations will determine the unknown forces acting at the crown of the arch for each half. These equations of condition for finding the unknown forces at the crown will now be written for a symmetrical arch. The general method is equally applicable to an unsymmetrical arch, however.

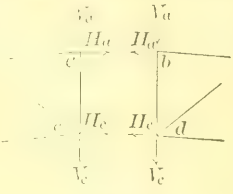

FIG. 4.

Let a load of 1 lb., at any point, x, cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 5.

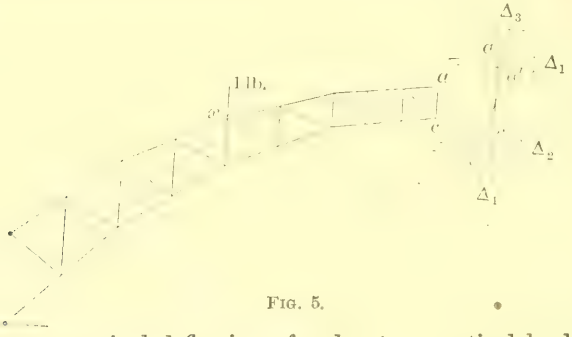

FIG. 5.

Then Δ_1 = vertical deflection of a due to a vertical load of unity at any point, x;

Δ_2 = vertical deflection of c due to a vertical load of unity at any point, x;

Δ_3 = horizontal deflection of a due to a vertical load of unity at any point, x;

Δ_4 = horizontal deflection of c due to a vertical load of unity at any point, x.

Let a vertical load of 1 lb. at a cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 6.

Then d_1 = vertical deflection of a due to a vertical load of unity at a;

d_2 = vertical deflection of c due to a vertical load of unity at a;

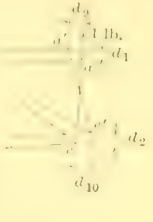

Fig. 6.

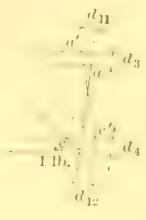

Fig. 7.

d_9 = horizontal deflection of a due to a vertical load of unity at a;

d_{10} = horizontal deflection of c due to a vertical load of unity at a.

Let a vertical load of 1 lb. at c cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 7.

Then d_3 = vertical deflection of a due to a vertical load of unity at c;

d_4 = vertical deflection of c due to a vertical load of unity at c;

d_{11} = horizontal deflection of a due to a vertical load of unity at c;

d_{12} = horizontal deflection of c due to a vertical load of unity at c.

Let a horizontal load of unity at a, as shown in Fig. 8, cause the points, a and c, to take the new positions, a' and c', respectively.

Then d_5 = horizontal deflection of a due to a horizontal load of unity at a;

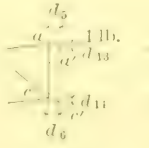

Fig. 8.

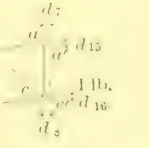

Fig. 9.

d_6 = horizontal deflection of c due to a horizontal load of unity at a;

d_{13} = vertical deflection of a due to a horizontal load of unity at a;

d_{14} = vertical deflection of c due to a horizontal load of unity at a.

Let a horizontal load of unity at c cause the points, a and c, to take the new positions, a' and c', respectively, as shown in Fig. 9.

Then d_7 = horizontal deflection of a due to a horizontal load of unity at c;

d_8 = horizontal deflection of c due to a horizontal load of unity at c;

d_{15} = vertical deflection of a due to a horizontal load of unity at c;

d_{16} = vertical deflection of c due to a horizontal load of unity at c.

The four simultaneous equations will now be written, from which the four unknown forces, V_a, V_c, H_a, and H_c, acting at the crown and due to a load of unity at any point, x, may be determined.

For the points, a and b, and the points, c and d, connected, the motion of a must equal the motion of b, and the motion of c must equal the motion of d.

The downward motion of $a = \Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15}$, and

the downward motion of $b = + V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15}$, and these must be equal, as they are for the same point.

Therefore,

$$\Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15} = V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15},$$

and

$$V_a d_1 + V_c d_3 = \frac{\Delta_1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The downward motion of $c = \Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16}$, and

the downward motion of $d = + V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16}$, therefore,

$$\Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16} = V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16},$$

and

$$V_a d_2 + V_c d_4 = \frac{\Delta_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The motion to the right of $a = \Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7$, and the motion to the right of $b = - V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7$, therefore,

$$\Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7 = - V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7,$$

and

$$H_a d_5 + H_c d_7 = - \frac{\Delta_3}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The motion to the right of $c = -J_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8$,
and the motion to the right of $d = +V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8$,
therefore,

$$-J_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8 = V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8,$$

and

$$H_a d_6 + H_c d_8 = \frac{J_4}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Solving these four equations, the following values for the unknown forces are found, in terms of twelve easily determined deflections:

$$V_a = +J_1 \frac{d_4}{2(d_1 d_4 - d_2 d_3)} - J_2 \frac{d_3}{2(d_1 d_4 - d_2 d_3)}$$

$$V_c = -J_1 \frac{d_2}{2(d_1 d_4 - d_2 d_3)} + J_2 \frac{d_1}{2(d_1 d_4 - d_2 d_3)}$$

$$H_a = -J_3 \frac{d_8}{2(d_5 d_8 - d_6 d_7)} - J_4 \frac{d_7}{2(d_5 d_8 - d_6 d_7)}$$

$$H_c = +J_3 \frac{d_6}{2(d_5 d_8 - d_6 d_7)} + J_4 \frac{d_5}{2(d_5 d_8 - d_6 d_7)}$$

If the values of J_1 to J_4, and d_1 to d_8, as found by actual computation, are in the direction heretofore assumed, their numerical value is to be inserted in the foregoing expressions without regard to sign, if they are found to be in a direction opposite to that assumed, they must be inserted with a minus sign.

From Maxwell's Theorem, it is known that:

$$d_2 = d_3, \text{ and } d_6 = d_7.$$

For any particular configuration of members in the arch rib, very much simpler expressions for V_a and V_c may be written.

For the case assumed, to illustrate this paper, a load at any panel point other than a,

makes $J_1 = J_2$, and $d_3 = d_4$, and therefore $V_a = 0$.

A load at panel point, a,

makes $J_1 = J_2 + (d_1 - d_2)$, and d_3 is still $= d_4$, therefore $V_a = \frac{1}{2}$.

For any panel point of the rib loaded except a

$$V_c = \frac{J_2}{2d_4}$$

And for panel point, a, loaded:

$$V_c = 0.$$

Restated, for any panel point loaded other than *a*

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}$$

And for panel point, *a*, loaded:

$$V_a = \frac{1}{2}, \text{ and } V_c = 0$$

For a configuration at the crown, as indicated in Fig. 10, that is, where the diagonals adjacent to the crown meet at the top of the crown vertical, $d_1 = d_2$, and d_3 and d_4 are no longer equal.

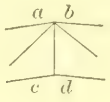

FIG. 10.

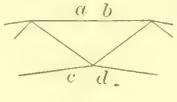

FIG. 11.

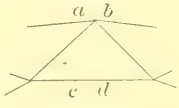

FIG. 12.

Then, for any panel point loaded other than *c*,

$\Delta_1 = \Delta_2$, and therefore,

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0$$

And for panel point *c*, loaded, $\Delta_1 = \Delta_2 - (d_4 - d_3)$, and $\Delta_2 = d_4$, and therefore,

$$V_a = 0, \text{ and } V_c = \frac{1}{2}$$

For such an arrangement of members at the crown as shown in Fig. 11, it is readily observed that

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}$$

for any panel point loaded.

For such an arrangement at the crown as shown in Fig. 12, it is seen that

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0$$

for any panel point loaded.

The expressions for the horizontal crown forces remain the same for all the cases selected.

Letting $2\ (d_5 d_8 - d_6^2) = n$, and remembering that $d_6 = d_7$, we have, for the value of the horizontal thrusts,

$$H_a = -\Delta_3 \frac{d_8}{n} - \Delta_4 \frac{d_6}{n}, \text{ and}$$

$$H_c = +\Delta_3 \frac{d_6}{n} + \Delta_4 \frac{d_5}{n}$$

The above expressions for the unknown crown forces have been written from assumed values of the twelve deflections of the half arch acting as a statically determined structure. For any actual problem, these deflections, of course, should be determined accurately, both as to direction and magnitude. The expressions for the value of the unknown crown forces will always be of the same general form as the above, but may differ in some of the signs of the terms.

The resulting plus values for V_a, V_c, H_a, H_c, indicate that they act as assumed, and the minus values that they act in opposite direction.

Temperature Stresses.—Let Fig. 13 show the motions of *a* and *b*, and *c* and *d*, for a rise in temperature when each half of the arch is considered as a separate structure, the full lines representing the normal figure and the dotted lines the figure changed by a rise in temperature.

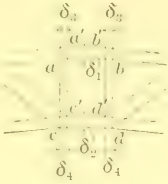

FIG. 13.

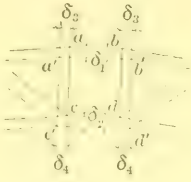

FIG. 14.

Let Fig. 14 show the motions of *a* and *b*, and *c* and *d*, for a corresponding fall in temperature under the same conception as before.

Then δ_1 = vertical deflection of *a* and *b* due to a certain change in temperature;

δ_2 = vertical deflection of *c* and *d* due to a certain change in temperature;

δ_3 = horizontal deflection of *a* and *b* due to a certain change in temperature;

and δ_4 = horizontal deflection of *c* and *d* due to a certain change in temperature.

Four equations for the unknown crown forces due to connecting the two halves may now be written for both a rise and a fall in temperature, using the notation as previously defined.

For a rise in temperature:

The upward motion of $a = + \delta_1 + V_a d_1 + V_c d_3 - H_a d_{13} - H_c d_{15}$, and the upward motion of $b = + \delta_1 - V_a d_1 - V_c d_3 - H_a d_{13} - H_c d_{15}$, for the two halves connected.

These motions must be equal, therefore, $V_a d_1 + V_c d_3 = 0$..... (5)

In an entirely similar manner, the three following equations may be written:

For a fall in temperature:

$$V_a d_2 + V_c d_4 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$
$$H_a d_5 + H_c d_7 = -\delta_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$
$$H_a d_6 + H_c d_8 = -\delta_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$
$$V_a d_1 + V_c d_3 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$
$$V_a d_2 + V_c d_3 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6a)$$
$$H_a d_5 + H_c d_7 = \delta_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7a)$$
$$H_a d_6 + H_c d_8 = \delta_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8a)$$

An examination of Equations (5), (6), (5*a*), and (6*a*), shows that V_a and V_c are zero, therefore, the only crown forces due to temperature are horizontal.

For a rise in temperature, remembering that $d_6 = d_7$, and making $(d_5 d_8 - d_6^2) = g$:

$$H_a = -\delta_3 \frac{d_8}{(d_5 d_8 - d_6^2)} + \delta_4 \frac{d_6}{(d_5 d_8 - d_6^2)} = -\delta_3 \frac{d_8}{g} + \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = -\delta_3 \frac{d_6}{(d_5 d_8 - d_6^2)} + \delta_4 \frac{d_5}{(d_5 d_8 - d_6^2)} = -\delta_3 \frac{d_6}{g} + \delta_4 \frac{d_5}{g}$$

For a fall in temperature,

$$H_a = +\delta_3 \frac{d_8}{(d_5 d_8 - d_6^2)} - \delta_4 \frac{d_6}{(d_5 d_8 - d_6^2)} = +\delta_3 \frac{d_8}{g} - \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = +\delta_3 \frac{d_6}{(d_5 d_8 - d_6^2)} - \delta_4 \frac{d_5}{(d_5 d_8 - d_6^2)} = \delta_3 \frac{d_6}{g} - \delta_4 \frac{d_5}{g}$$

The foregoing gives a simple method for finding the stresses in every member of the arch due to a load at any point, or for any desired change in temperature. Under this method, the work may be easily divided into several sets of independent and easily understood operations at the very beginning of the computations, thereby leaving only the small and easily managed theoretical part for the person in charge.

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PAPERS AND DISCUSSIONS

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WALNUT LANE BRIDGE, PHILADELPHIA

By George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E.

To be Presented September 15th, 1909.

The bridge on the line of Walnut Lane, Philadelphia, over Wissahickon Creek, is notable for the great size of its main arch, for the novelty of some of its features, the character of its material, and the method of its construction. It is hoped that a description of it, with a discussion of the principles of its design, the reasons for its special features, and the lessons learned in the course of its construction will be of interest to the Profession.

The ravine of Wissahickon Creek, in Fairmount Park, is narrow and deep. It separates two populous sections of the City—Germantown and Roxborough—which are both on high ground, and heretofore most of the travel between them has been required to make a long detour, all the bridges being at a low level and the approaches to them too heavy in grade for business hauling.

By Ordinance of July 13th, 1905, City Councils authorized the construction of a high-level bridge on the line of Walnut Lane, the point where the ravine is narrowest and most advantageous for the crossing.

The required length of structure, of course, depended on the elevation of the deck, and as considerable grading of approaches was neces-

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sary—cutting for one and filling for the other—the rate of grade to be obtained on the approaches, together with the estimated money value of each percent. of reduced street gradient, entered into the consideration of the problem of dimensions and design. The question was somewhat further involved by the fact that on the Germantown side the approach is from three streets. Fig. 1 exhibits the original profiles and the elevations and grades adopted. A portion of the filling for the west approach was obtained from the cut of the east approach, and was carted across the bridge.

The bridge deck was given an ascending grade to the east—1½% to the middle of the main span and 1% from there to the east abutment. This change of grade was made in order to produce a slight crowning and relieve the elevation, or side view of the bridge, of the optical illusion of a dished appearance in the deck over the arch. This crowning is equivalent to a local grade of 1 in 400 on each side.

The grades adopted give the floor at the middle an elevation of 147 ft. above ordinary water level in the creek. As the depth of the structure at the crown is 14 ft., the soffit is 133 ft. above the creek and about 103 ft. above the drive, giving an imposing and majestic sweep of arch that is very impressive.

As the two springing points are at the same elevation, the grade over the bridge, in making the spandrel piers on one side higher and therefore thicker, puts extra dead load on that side, and this is counterbalanced by making the walls of the spandrel piers on the other side 4½ in. thicker.

In considering the matter of type of structure and length of spans, it was regarded as very undesirable, for æsthetic reasons, that there should be any pier or column interposed between the creek and the pleasure drive which winds along the west bank at an elevation of about 30 ft. above the water. The situation clearly indicated an arch as the only suitable form of support for the roadway, and the necessity of clearing both the drive and the creek, while making reasonable provision for the future widening of the drive, taken in consideration with the profile of the rocky bluffs, fixed the location of the abutment piers and the span of the main arch, which was made 233 ft. between the vertical faces of the piers.

The extraordinary length of the main span gave to the structure a monumental character, and it became important that the remainder of

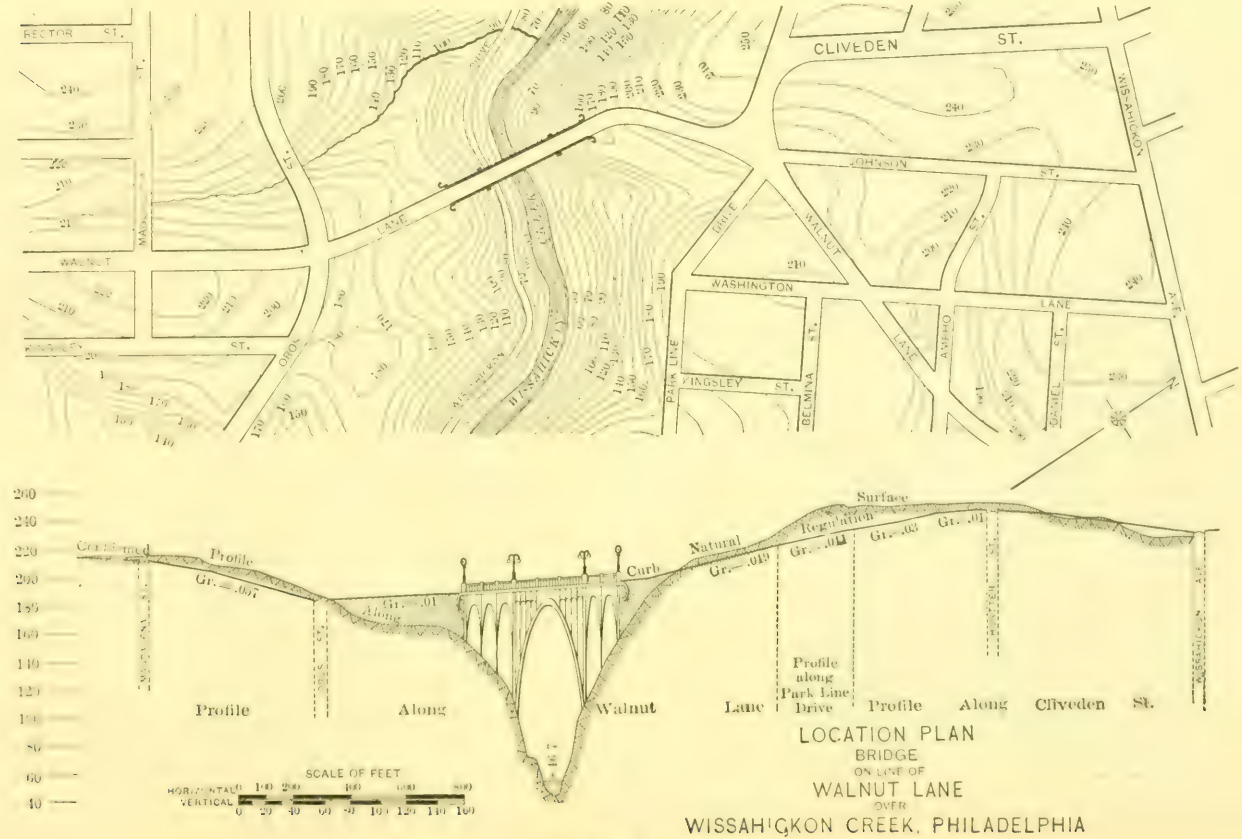

RECTOR ST.
CLIVEDEN ST.
WISSAHICKON
JOHNSTON ST.
WALNUT
WASHINGTON LANE
KINGSLEY ST.
PARK LINE
BELMINA ST.
AMPHO
DANIEL ST.
Profile Along Walnut Lane
Profile along Park Line Drive
Profile Along Cliveden St.
Surface
Regulation
Natural
Curb
Gr. .007
Gr. .01
Gr. .03
Gr. .04
Gr. .049
SCALE OF FEET
HORIZONTAL
VERTICAL
LOCATION PLAN
BRIDGE
ON LINE OF
WALNUT LANE
OVER
WISSAHICKON CREEK, PHILADELPHIA

FIG. 1.

the design and the material of which the bridge was to be constructed should be in accord with it.

The lowest first cost would have been found in a steel viaduct with a wooden floor, but the character of the surroundings—natural park scenery of rocky and wooded slopes—would cause such a structure to appear inappropriate and inadequate, and the possibility of future neglect of proper maintenance enhanced the importance of selecting a construction of the most permanent character.

The development of the sphere of usefulness of concrete, and the city's extended experience in the use of it for arches, walls, and floors, with new and improved methods of manipulation and construction, whereby more nearly uniform character and value are obtained, and the adequacy and permanence of the material are assured, together with the great economy obtained through its use, influenced the adoption of concrete as the structural material of the bridge throughout, and it was designed so as to require steel reinforcement at only a few points.

The most economical length of the approach spans was determined by trial to be 60 ft. from center to center of the piers, or 53 ft. clear. The conditions called for three of these spans on the west slope and two on the east slope of the ravine.

The profile of the rocky banks naturally fixed the location of the springing points of the main arch, and gave a graceful proportion between the clear height above the creek and the length of the span.

The type of structure selected is that of the stone arch bridge in Luxemburg, Europe, built in 1904, which is substantially two bridges side by side with the space between them floored over. The type is economical for such a structure, because large masonry arches of a width sufficient for their own independent stability have more than ample strength for any highway live load that may have to be carried, and any increase in width or strength would be utilized only in sustaining the weight of the added material. Thus, in the main span of this bridge, the arch ribs themselves constitute one-half of the whole load to be carried—both dead and live—and, as the live load is less than one-twelfth of the dead load, a single barrel of the full width of the floor would clearly be wasteful because its thickness could not safely be made much less than that of the narrow ribs. The width of the floor is 60 ft., while that of each of the two ribs is 18 ft.—giving

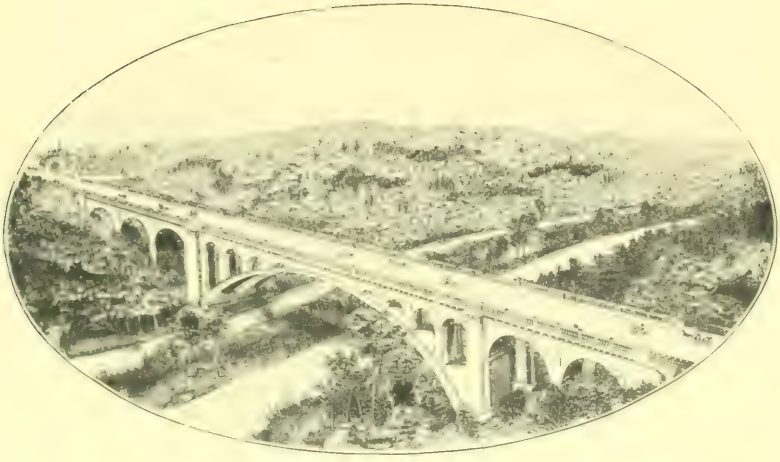


FIG. 1.—WALNUT LANE BRIDGE OVER WISSAHICKON CREEK, PHILADELPHIA.

FIG. 2.—NORTH SIDE, WALNUT LANE BRIDGE.

together 36 ft. of supporting arch, or only 60% of the floor width. The space between the ribs is 16 ft., leaving 4 ft. of floor to overhang on each side.

Each of the two halves or ribs of the bridge carries two vertical walls, 2 ft. 6 in. thick, which extend up to and carry the floor. These walls are set in 9 in. from the edges of the ribs, giving an actual overhang of 4 ft. 9 in. to the sidewalk floors on the outside of the bridge.

The three aisles thus formed, and spanned by the floor, are 20 ft. and 14 ft. for the middle and side aisles, respectively, from center to center of the walls.

These longitudinal walls are full spandrels over the 53-ft. arches, but on the main arch they are pierced by four 20-ft. arches over each haunch, leaving 60 ft. blank over the crown. These 20-ft. openings are flanged 6 in., in order to give the barrel or rib of each arch a width of 3 ft. 6 in., and pilasters of equal relief, 2 ft. 10 in. wide, were made between the arches to afford angles for concealing the vertical joints in the wall. In the 53-ft. arches the flanging of the walls is 9 in., making the ribs 4 ft. wide, but a sheeting of concrete, 18 in. thick, connects the ribs into pairs constituting the barrels, 18 ft. wide.

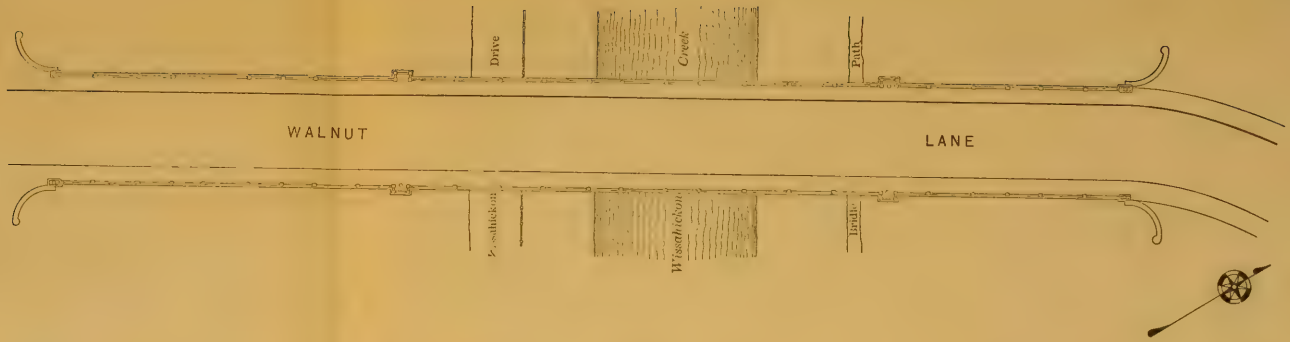
For architectural effect, the abutment piers at the ends of the main arch are widened to project 3 ft. beyond the general line of the structure, and they provide for refuge bays in the sidewalks of the deck. The twin shafts of these piers are hollow, 6 by 10 ft., giving a thickness of wall of 3 ft., which is increased below the water-tables by the pilasters under the arches. They have a floored chamber under the sidewalk, in order to give access to the electrical conduits and the bases of the light poles. The 10-in. vertical wrought-iron drain pipes from the grate-top inlets in the driveway gutters are secured to the inside wall of these shafts, and pass the drainage out through 16-in. cast-iron pipes built into the body of the pier and outletting on the rocky banks of the creek.

The floor structure consists of transverse rolled-steel I-beams, 6 ft. from center to center, with concrete jack-arches between them. The beams over the 20-ft. middle aisle are 20-in. 65-lb. per ft., and those over the 14-ft. side aisles are 15-in. 42-lb. per ft. The size of the floor members was determined by the assumed concentrated loads adopted as the standard for highway bridges in the City of Philadelphia, which consist of a truck, weighing 40 tons, on four wheels

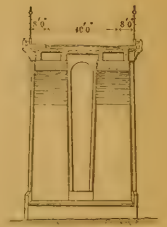
spaced 20 ft. between axles, and 6 ft. gauge, or two street railway cars side by side, weighing 20 tons on each truck.

The concrete jack-arches can be depended on to distribute an axle load over two or more beams when the axles are more than 6 ft. apart, for a beam cannot deflect under its load without throwing some load on its immediate neighbors. The floor concrete is filled monolithically to 3 in. above the tops of the floor-beams. If the spring of the jack-arch be taken at the lower fillet of the beams, and a centroid of pressure be assumed 1½ in. above the tops of the beams, there is constituted for distributing purposes an arch of 12-ft. span, having a rise equal to the depth of the beam, and an effective thickness of 3 in. at the crown. A further assumption is made that the embedded beam will distribute a concentrated wheel load of 10 tons laterally over a space equal to the gauge of the wheels—6 ft. Each transverse foot of floor arch, therefore, is required to sustain 3 333 lb. of live load, and if one-half of this be distributed to the adjacent beams, the horizontal force developed in the distributing arch will give, in the worst case—that of the 15-in. side aisle beams—a unit stress of only 112 lb. per sq. in. Thus it appears unnecessary to compute the beams for more than one-half of an axle load when the axles are 12 ft. or more apart, but the standard weight of the beams used gives value equivalent to five-eighths of the 20-ton axle load at a fiber stress of 15 000 lb., without allowance for the reinforcing value of the encasing concrete. As it is found by experiment, confirming calculations based on conservative assumptions of adhesion of the concrete to the steel, that the embedment of rolled beams in concrete adds one-third to their carrying capacity, the margin possessed by the floor to carry heavier live loads than those specified is so great that the live load may be doubled without stressing the steel beyond common practice. The construction proves to be very rigid under rolling loads.

Careful estimates were made of the relative cost of floor construction of this type and of reinforced concrete—concrete beams and slabs reinforced with steel bars. The latter appeared to be the lower in cost by about $4.60 per panel, or less than $1 per lin. ft. of bridge, but at that time experience with construction of that class, for the severe service of heavy street traffic, was limited, and it was felt that the steel beams with independent strength were less of an experiment, and besides, the beams would furnish support for the forms of the floor



SECTION *C-D*
LOOKING WEST



SECTION *A-B*
LOOKING WEST

DATA:
Main span, 233 ft. clear.
Rise of main arch, 70 ft. 6 in.
Approach spans, 53 ft. clear.
Spandrel arches, 20 ft. clear.
Total length of bridge, 585 ft.
Height above creek, 147 ft.

CONCRETE BRIDGE
ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
PHILADELPHIA

Scale of Feet
20 0 20 40 60

arches. As these forms were required to be carried high in the air, the support afforded by the beams, and consequently the lesser cost of scaffolding, doubtless quite offset the higher cost of the material.

The 15-in. beams and the 20-in. beams in line with them were firmly connected by heavy angle irons with four $\frac{7}{8}$-in. turned bolts, in order to constitute a tie between the two ribs of the arch, and prevent any possibility of a movement apart and split of the floor.

All the beams are completely encased in concrete, which is 3 in. thick over the top surface and around the bottom flanges. At the latter point, $\frac{3}{16}$-in. wire loops, 3 in. apart, are used to prevent the concrete from loosening.

The floor over the middle aisle is depressed to permit the use of wooden ties and cinder ballast for two street-railway tracks, and to afford also, if it should ever be required, depth for an underground trolley conduit.

The whole of the driveway floor surface below the paving is waterproofed with a coating of coal-tar roofing pitch from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick.

The depresssion is filled with cinders, over which is laid the 6-in. concrete paving base. The driveway, 40 ft. wide between curbs, is paved with asphalt, and the two sidewalks, 8 ft. 3 in. wide, are granolithic with steel-edged curbs.

The 2½-ft. longitudinal walls are connected at intervals over the piers and ribs of the approach arches, and over the ribs of the main arch, by 24-in. cross-walls, which serve as braces for the walls and as diaphragms or transverse bracing to transmit to the arch ribs the wind forces on the floor. The floor is separated by joints into sections to localize the contraction, and the interruption of its continuity prevents its action as a horizontal beam to transmit the wind stress to the main piers. Each section, therefore, is required to transmit its horizontal forces to the arch ribs. To provide the requisite stability in the long slender ribs of the main arch, they are flared 1 ft. 9 in. on each face, from the crown to the spring, increasing the width from 18 ft. at the crown to 21 ft. 6 in. at the spring. This flaring corresponds to a vertical batter on each face of 1 in 40 in a projected cross-section, and the edges of the ribs, therefore, are in planes parallel to and almost coinciding with the battered faces of the piers. It also causes a slight but not noticeable warp in the face surface, the ribs being made rectangu-

lar in section at every point in a radial direction. The thickness of the ring is 9 ft. 6 in. at the spring and 5 ft. 6 in. at the crown. The ribs of the approach spans are not flared—the width being 18 ft. throughout—and their faces are plumb.

The transverse walls do not extend over the space between the ribs, the latter being independent of each other, but the shafts of the piers at the ends of the main arch are connected by a transverse arch of 12 ft. clear span and 10 ft. wide, having embedded in it twelve 1-in. square steel bars diagonally crossed to act as transverse bracing for the shafts. The transverse walls on the ribs are 2 ft. thick throughout the bridge, except those over the west half of the main arch where they were increased in thickness to 2 ft. 4½ in., in order to provide sufficient weight to make the dead load on the arch symmetrical by counterbalancing the heavier load on the east half caused by the greater height of the spandrel piers due to the ascending grade over the bridge.

The abutments were especially designed to economize material and at the same time avoid too utilitarian an appearance. The thrust of the end arch ribs is utilized to balance the earth pressure; and the curvature of the wings, besides embellishing the structure, has the effect of broadening the foundation and thus giving the equivalent of a much thicker wall to resist overturning. This effect is increased by the construction of a buttress, 4 ft. thick, at the extreme end of the wing in a direction radial to the curve, or at an angle of 45° with the axis of the bridge. At the junction of the wings and the front wall, steel rods are embedded for the double purpose of giving cantilever value to the projecting wings, and preventing vertical cracks in the sharp re-entrant angles of the corners. Some of these rods are continued in a straight line past the middle of the curve of the wing and within 6 in. of the surface, in order to give, if it should be required, beam value to the wing between the abutment wall and the buttress at the end of the wing.

The east abutment is 64 ft. high, from foundation to coping, about 38 ft. of this being above the natural surface of the ground in front of it. The south wing of this abutment is built around a 30-in. water main, a hole 6 ft. in diameter being made in the wing to give free access to all sides of the pipe for repairs. The foundations of all the piers were required to be constructed over this pipe, and in each case an arch was formed over it to give 18 in. clear space around it, steel

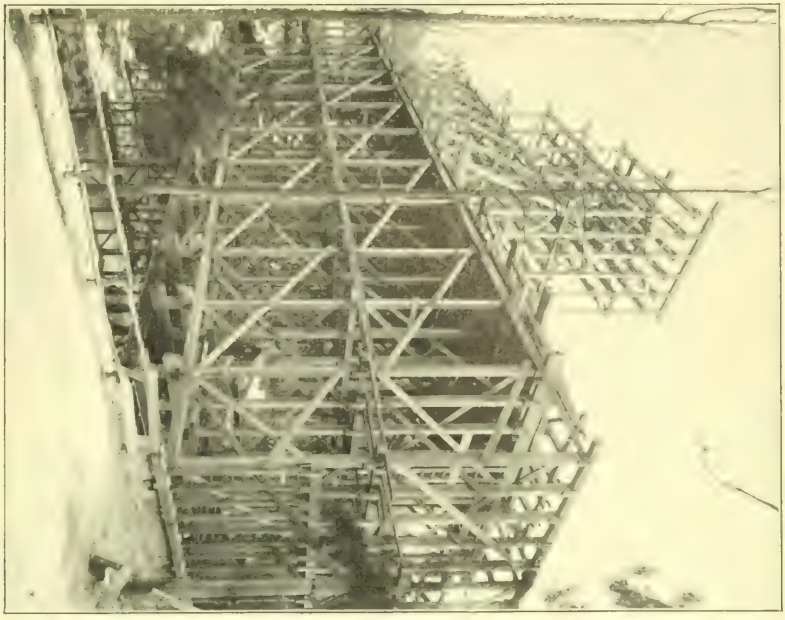


FIG. 1.—FALSEWORK, WALNUT LANE BRIDGE.

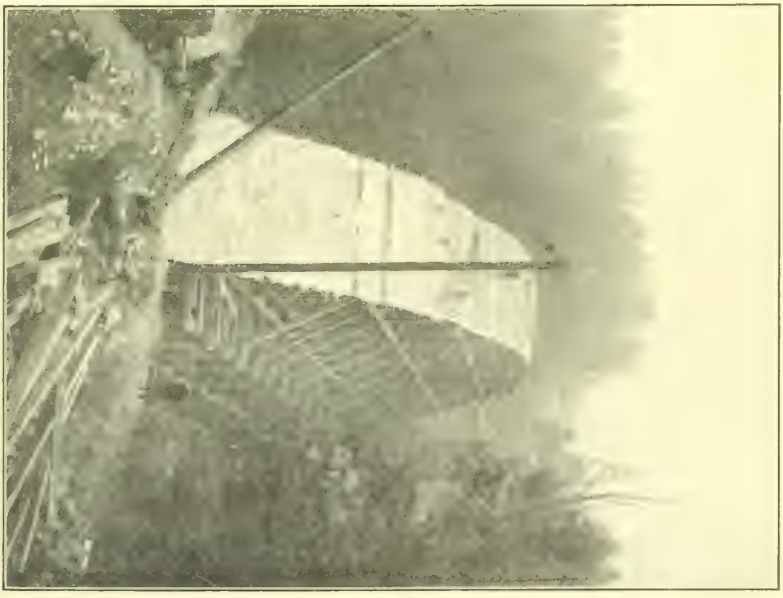


FIG. 2.—MAIN ARCH CENTERING, WALNUT LANE BRIDGE.

rods being embedded over the openings to resist any tendency to crack. The foundation of Pier 4 was extended because the pipe arch was near the end of the pier.

All piers, abutments, and wings, were founded on solid rock, the surface of which was found to be extremely rugged, requiring no stepping. In the foundations of each of the main arch piers two drill holes were sunk 8 ft. below the bottom to make sure of the solidity of the rock, and no crevices were found. The rock is a mica schist.

The open character of the spandrels on the main arch, with their numerous vertical joints, presented a condition quite different from that commonly found with solid spandrels of rigid walls and earth filling, where the load is distributed and the tendency to buck up is met with concentrated resistance. Here, the loads, except immediately over the crown, are all concentrated, and there is no stiffness in either walls or floor to concentrate resistance to buckling. Each panel delivers its own individual load, dead and live, at the same point always, and resists to only the same amount. The long ring rib, therefore, acts as a column, square-ended, throated in the middle, subject to varying eccentric loads, and made up of blocks accurately fitted together but not adhering, and, consequently, it is dependent for its stability entirely on its sufficiency of thickness to accommodate the varying center of pressure. Therefore, it became specially important to determine the extreme eccentricity of the pressure line by trying every possible condition of loading, both concentrated and distributed. The effects of six different conditions are shown by the stress diagram, Plate XLIV. The shape adopted for the arch is an intermediate between the extremes of eccentricity, coinciding very nearly with the line of dead or constant loading, with a slight arbitrary allowance at the spring for possible initial stress in the intrados from any shrinkage or settlement which might take place after keying, or from being keyed at a temperature above mean, as seemed likely to be the case. This shape was obtained by compounding segmental curves on three centers for each surface, as shown on Plates XXXIII and XLIV.

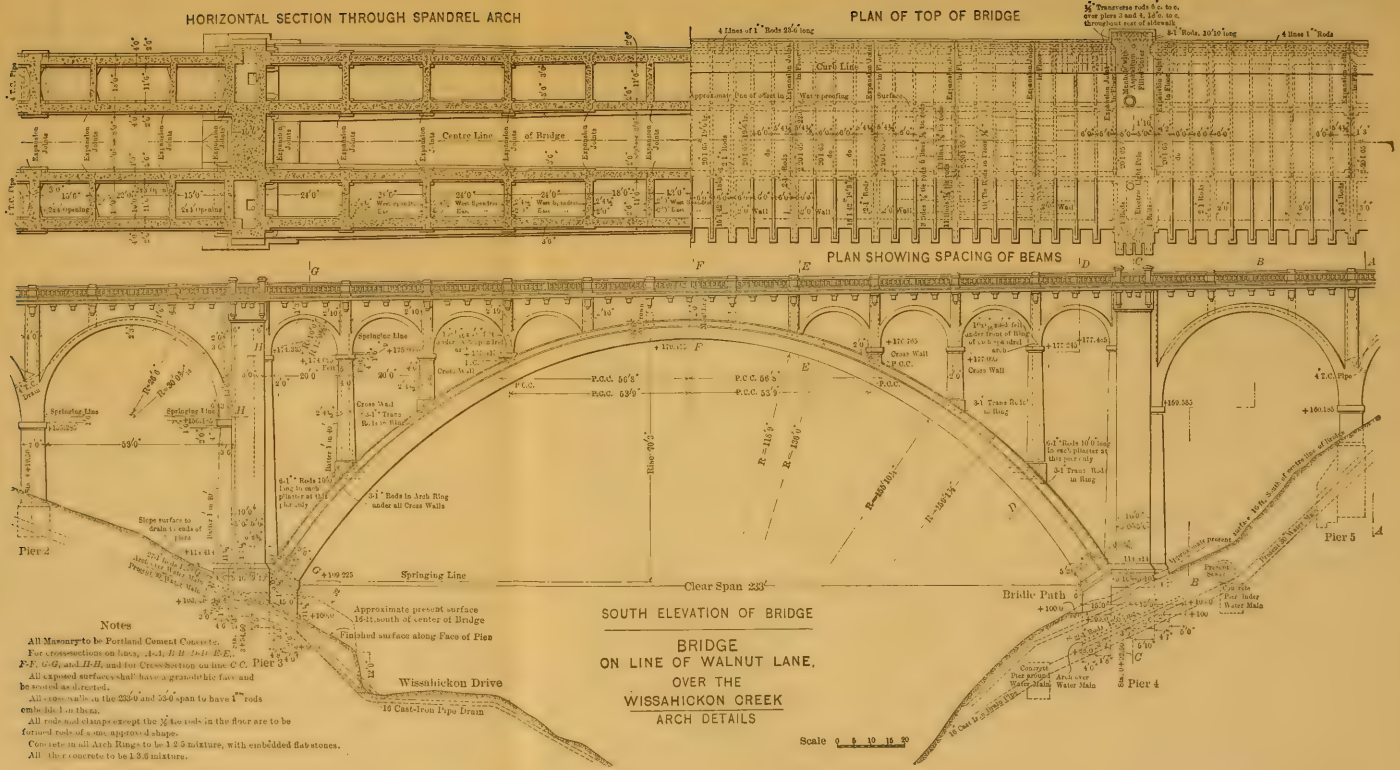
The arch was made square-ended, and continuous at the crown, in preference to hinging it. While pivotal bearings at the three points would have eliminated temperature stresses and reduced the likelihood of initial stresses from falsework settlement, the cost would have been

greater, and far less graceful lines would have resulted. The line of center pressure, if confined to pivots at springs and crown, would depart far from the mid-line of the ring at the quarters and require considerably greater thickness there than is needed with the flat-end bearing. The continuous taper in thickness as well as width of rib, from the abutments to the crown, is an important element of beauty in the design. In further pursuit of lightness in the appearance of the ring, a simple moulding was constructed on the outer face of each rib, tapering in width in proportion to the thickness of the ring.

The span of the arch, from center to center of bearings on the skewbacks, is 240 ft., and the rise of the mid-line is 70.5 ft.

The maximum fiber stress appears to be at the spring, where there is the greatest theoretical eccentricity, because of the arbitrary allowance made for initial stress. This maximum is 356 lb. per sq. in., and the average at the same point is 167 lb.

The temperature stresses were computed in two ways, the results corresponding fairly. One method was by using the ordinary deflection formula for beams, and the other was by calculating the increase and decrease in length of extrados and intrados on the curvature resulting from the change in length of the ring and its confinement at the ends, the changes in length of fiber representing stress assumed to be in proportion to a modulus of elasticity of 2 000 000 in both compression and tension. Computations were based on a change of the general temperature of the arch of 40° Fahr., either way from the temperature at keying, which will change the length of the arch 1 in. and cause a change of 1 in. in the elevation of the crown. A rise increases the curvature at the crown, shortening its radius, and—because of the fixed ends—decreases the curvature of the haunches, lengthening the radius. This produces compression of the intrados at the crown—maximum at the middle—and of the extrados at the haunches—maximum at the spring—the effect being zero at the point of contra-flexure. This point was assumed to be at the *K* line of the stress diagram, Plate XLIV, which was computed by the Cain method, 82 ft. from the spring, and the substantial correctness of the locality was demonstrated by the close agreement of the bending stresses in the several divisions, the deformation of the arch being necessarily an automatic adjustment of the shape to produce uniformity of fiber stress in bending.



BRIDGE ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK

ARCH DETAILS

Measurements of the changes in elevation of the crown since the completion of the arch show a total difference between the extremes of winter and summer of 1⅛ in. which is equivalent to a change in length of arch ring of 1⅛ in. and a change in general temperature of the concrete of 45° Fahr. The actual temperature of the arch at the time of keying is not known for either rib. It is probable that the first one, because of some heat from crystallization and the atmospheric warmth of the mid-June season, was within 10 or 15° of the highest to be reached, and the second rib, keyed in November, was probably very near the mean temperature.

Some time after the completion of the arch rings, an electric resistance thermometer, of the standard type made by Leeds and Northrup, was embedded in one of the ribs on the mid-line near the spring, and weekly observations were made of the temperature of the heart of the concrete. A diagram showing the results of the changes in temperature and in the elevation of the crown of the arch is shown on Fig. 2. The heavy upper line gives the actual temperature in the arch; the dotted lines show the maximum, minimum, and mean weekly temperatures, as given by the United States Weather Bureau station in Philadelphia. The solid lower lines indicate the elevation of the two arch ribs. It will be observed that the total change of temperature in the center of the arch amounts to 42.8° Fahr., which gives a close check on both the assumptions used in the design, and on the computations of temperature changes made from the change in elevation. The close relation between the various curves in the diagram is also evident.

The concrete of the structure was of several classes, but in all classes—foundations, piers, abutments, walls, and balustrades—stones of as large size as conveniently workable were embedded. Some derrick stones were used in the foundations, but did not appear to be economical, because, under the specifications, a sufficient depth of concrete was required to be in place to half embed the stone, and it was found impracticable to get in as large a volume of embedded stone as by the use of one-man size. Besides, the monopoly of a derrick, during the time of placing a large stone, delayed the delivery of material and reduced the yardage of work accomplished. The quantity of stone thus embedded varied in different portions of the work, according to circumstances—the consistence of the concrete, the character of the sand

and coarse aggregate, the convenience of location, and the skill of the workmen. Observations of the relative quantities of stone and mixer concrete used have determined that in horizontal courses, such as are laid in piers and walls, the embedded stone constitutes from 30 to 40% of the total volume of the finished work, depending on the shape of the stone and the efforts of the laborers. The percentage secured in the arch, where all the stones were required to be placed radially, was from 25 to 30. The contractors were encouraged to use as large a percentage as could be embedded and fully flushed in the concrete, and they were of course desirous of using as much as possible, because its cost was less than that of the concrete displaced by it.

The use of this embedded stone is found to be economical in almost all classes of concrete work, even such as is reinforced with steel rods. Of course, care must be taken in reinforced work that stones do not ride upon rods in a manner to permit the formation of voids under the stones. The increase in compressive strength of all classes of concrete obtained by the embedment of large stones, as indicated by the tests of cubes, enhances the value of the material for almost if not quite all classes of work. The stones also very materially increase the shearing strength of the concrete, and generally reduce its cost.

The concrete of the ring of each 53-ft. arch was deposited continuously, was accomplished always in one day or less, and is of the same character and quality as that in the main arch—1:2:5 mixer concrete with flat stone embedded radially. The skewback was formed at the point of meeting of the arch ring with the transverse walls on the piers, the portion of the arch below this—called the umbrella—being constructed as a part of the pier.

The arches of this bridge, as constructed of this composite material, may be regarded as stone arches of a grade of workmanship superior to the ordinary rubble arch, because of the complete bedding of the individual stones. The increased compressive strength obtained by the embedment of the stone is attended, of course, with a considerable increase in the shearing resistance, and this quality is valued in consideration of the forces operating in the arch at the skewback, where eccentricity of direct stress produces actual shearing stress. The main arch ribs are practically columns, with more or less eccentric loading, and with varying and somewhat indefinite intensity of bearing stress at the ends. Concrete prisms or columns, when compressed to destruc-

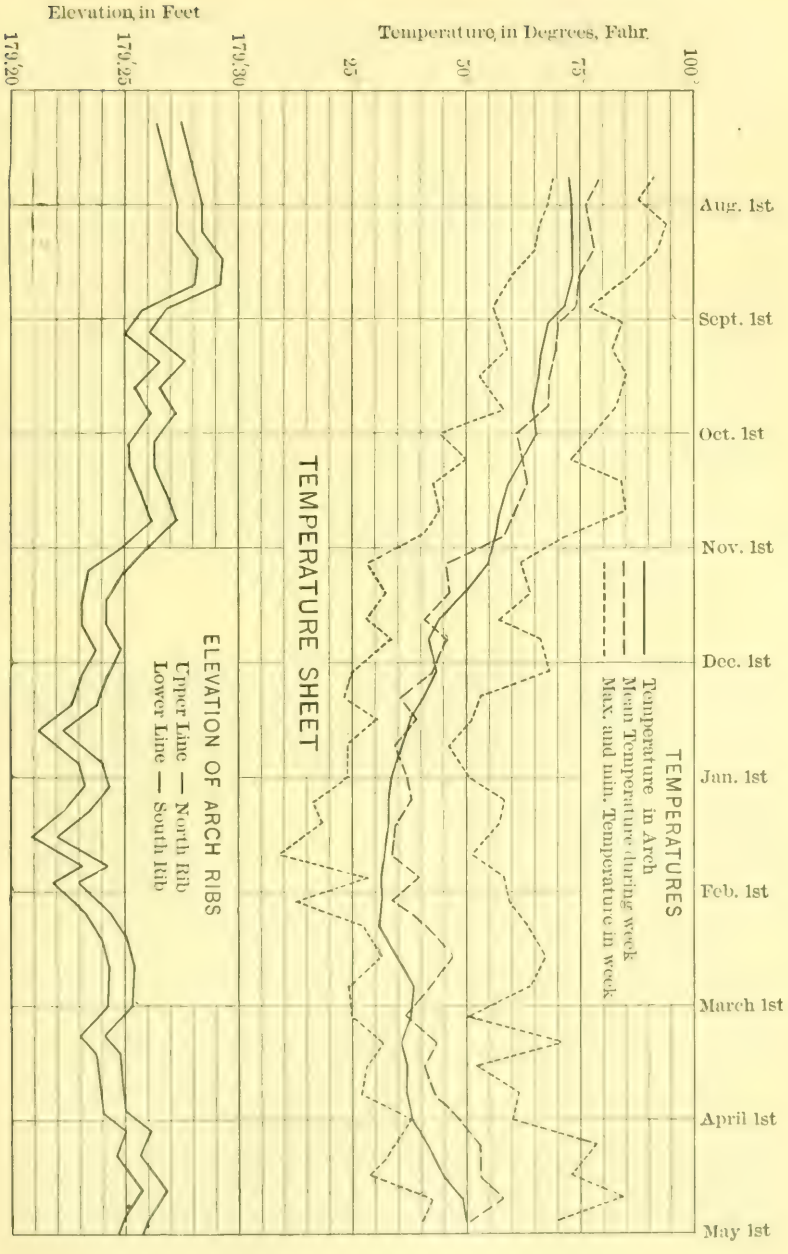

Elevation in Feet
179.20
179.25
179.30
Temperature in Degrees, Fahr.
25
50
75
100
Aug. 1st
Sept. 1st
Oct. 1st
Nov. 1st
Dec. 1st
Jan. 1st
Feb. 1st
March 1st
April 1st
May 1st
TEMPERATURE SHEET
ELEVATION OF ARCH RIBS
Upper Line — North Rib
Lower Line — South Rib
TEMPERATURES
Temperature in Arch
Mean Temperature during week
Max. and min. Temperature in week

Fig. 2.

tion, usually fail by shear, the angle approximating 39° with the axis of compression, and a force acting at any angle to the direction of the compression that causes the shearing will constitute a shearing force and hasten the failure. Such an angular force is produced by eccentricity of direct stress or by increments of loading and own weight. Tests of concrete model arches, made in order to determine questions of design, indicated that it is wise to treat the components of the direct arch stress at the abutment as so much shear acting in a beam. Regarded as such, the vertical component or reaction at the spring of the main arch ribs of this bridge amounts to 108 lb. per sq. in. of arch section in shear at the face of the skewback, which would be high for simple concrete, but is allowable for the hybrid material.

The mix of concrete used in the foundations and in the body of the piers and walls was 1 part Portland cement, 3 parts sand and 6 parts crushed stone. In the arches—the main and approach ribs and floor jack-arches—the mix was 1:2:5. The facing concrete or granolithic mixture of all showing surfaces, and also the body of the balustrade work was 1:2:3. It was all mixed so mushy that the large stone could easily be submerged in it by joggling.

The sand was required to be graded—grains varying from fine to coarse—and a yellow bank sand from New Jersey was accepted and used. Crushed stone was required to be graded in size from ¼ in. up, limited to 1½ in. generally, and to ¾ in. for arch and floor work. After using the 1½-in. size for a time the contractor preferred to use a 1-in. size limit because of the greater facility which it seemed to provide for embedding the large stone. The difficulty, however, with the 1½-in. stone was in obtaining a satisfactory grading—a proper admixture of all sizes from ¼ in. up—the practice of the quarries generally being to separate the 1½-in. stone from the smaller size. If, however, the run of the crusher up to the 1½-in. size can be obtained, the larger size will give equally satisfactory results and perhaps somewhat better concrete. The most thoroughly graded aggregate is found to produce, not only the densest and strongest concrete, but the smoothest working.

Every car lot of cement received was sampled after delivery at the work, and was tested in the municipal laboratory before being used. It was required to meet the standard specifications of the Bureau of Surveys of the City of Philadelphia, which are as follows:

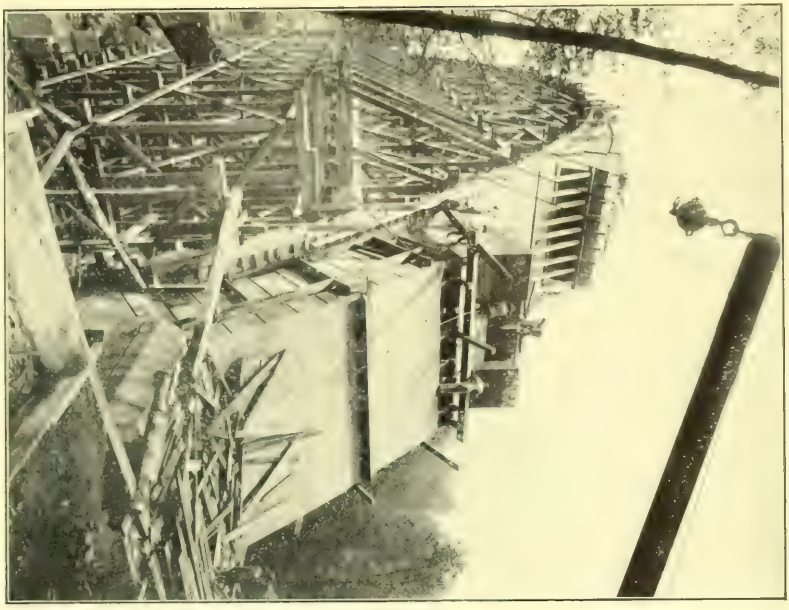


FIG. 1. WASHING THE SURFACE OF THE ARCH.

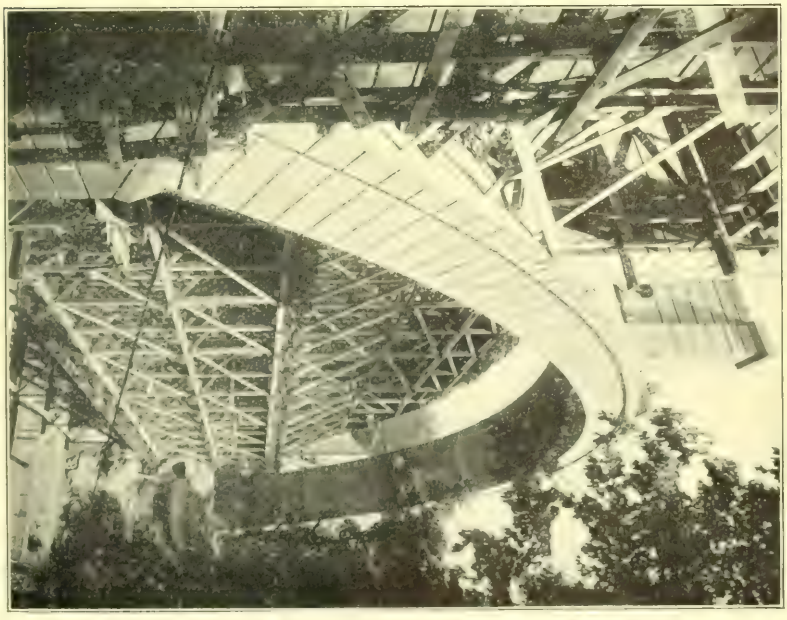


FIG. 2.—SOUTH RIB, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

"The acceptance or rejection of a cement to be used shall rest with the Chief Engineer, and will be based on the following requirements:

"Specific gravity not less than 3.1.

"Ultimate tensile strength:

7 days (1 day in air, 6 days in water) .....500 lb.
28 days (1 day in air, 27 days in water) .....600 lb.
7 days (1 day in air, 6 days in water) (1) one part cement to (3) three parts of standard quartz sand.....170 lb.
28 days (1 day in air, 27 days in water) (1) one part cement to (3) three parts of standard quartz sand.....240 lb.

"Fineness:

Residue on No. 100 sieve not over 8 per cent. by weight.
Residue on No. 200 sieve not over 25 per cent. by weight.

"Set: It shall require at least 20 minutes to develop initial set, and shall develop hard set in not less than 1 hour nor more than 10 hours. These requirements may be modified where the conditions of use make it desirable.

"Constancy of Volume: Pats of cement 3 inches in diameter, one-half inch thick at center tapering to thin edge, immersed in water after 24 hours in moist air, shall show no signs of cracking, distortion or disintegration. Similar pats in air shall also remain sound and hard. The cement shall pass such accelerated tests as the Chief Engineer may determine.

"Sulphuric anhydride (SO_3); not more than 1.75 per cent.

"Magnesia (MgO); not more than 4 per cent.

"Briquettes for testing shall be 1 sq. in. area of cross-section; sieves shall be of brass wire-cloth having approximately 9 800 and 37 500 meshes per square inch, respectively, the diameter of the wire being 0.0045 and 0.0023, respectively."

The accelerated test used consisted in making a small ball of neat cement paste, allowing it to harden 24 hours in moist air, and then boiling it for 3 hours.

For purposes of record and comparison, specimen 6-in. and 12-in. cubes of concrete, as taken from the mixer, were made by the inspectors at frequent intervals during the progress of the work, and tested in the city's laboratory. Standard briquettes were also made by the inspectors from the mortar extracted from the concrete. Table 1 shows the results of tests of both cubes and briquettes. A number of the cubes had embedded flat stones representing as nearly as practicable the character of the concrete made in the work.

	1 544	.....	278	2 944	1 903	505	2 317	2 000	485	.....	.....	....	4 928	.....	304	.....	.....	....
	(6)1 325	.....	176	2 029	.....	306	2 391	1 757	379	.....	.....	....	2 917	.....	448	.....	.....	....
	2 228	.....	338	2 678	.....	336	2 436	2 486	466	.....	.....	....	5 164	.....	423	.....	.....	....
	1 792	.....	472	2 172	.....	372	2 401	.....	341	.....	.....	....	4 143	.....	200	.....	.....	....
	2 592	.....	325	1 698	.....	208	3 006	.....	307	.....	.....	....	3 770	.....	213	.....	.....	....
	2 225	.....	332	2 107	.....	373	2 286	.....	305	.....	.....	....	5 369	.....	329	.....	.....	....
	2 015	.....	310	2 636	.....	617	2 406	.....	306	.....	.....	....	4 519	.....	337	.....	.....	....
	2 206	.....	357	2 381	.....	409	2 574	.....	297	.....	.....	....	4 469	.....	382	.....	.....	....
	3 046	.....	475	2 235	.....	319	2 586	.....	420	.....	.....	....	4 167	.....	454	.....	.....	....
	2 415	.....	447	907	.....	123	3 143	.....	476	.....	.....	....	4 723	.....	473	.....	.....	....
	.....	.....	....	2 153	.....	334	3 617	.....	433	.....	.....	....	5 937	.....	485	.....	.....	....
	.....	.....	....	(8)1 300	.....	290	3 188	.....	421	.....	.....	....	4 981	.....	528	.....	.....	....
	.....	.....	....	2 944	.....	645	3 102	.....	162	.....	.....	....	3 619	.....	258	.....	.....	....
	.....	.....	....	3 349	.....	624	3 436	.....	437	.....	.....	....	3 996	.....	649	.....	.....	....
	.....	.....	....	3 834	.....	695	.....	.....	....	.....	.....	....	3 109	.....	514	.....	.....	....
	.....	.....	....	3 147	.....	526	.....	.....	....	.....	.....	....	3 494	.....	503	.....	.....	....
	.....	.....	....	2 583	.....	289	.....	.....	....	.....	.....	....	3 101	.....	477	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	3 065	.....	444	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	2 153	.....	336	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	1 972	.....	275	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	4 021	.....	519	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	4 207	.....	554	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	3 879	.....	422	.....	.....	....
Average.....	1 943	1 748	354	2 648	1 942	420	2 893	2 061	441	3 158	2 288	377	3 657	2 704	393	3 350	.....	410
1:3:6 (2).....	1 784	2 150	336	2 851	2 958	652	.....	3 229	....	.....	2 695	....	.....	3 292	....	.....	.....	....
	.....	2 715	....	.....	3 417	....	.....	3 236	....	.....	2 764	....	.....	3 166	....	.....	.....	....
	.....	2 347	....	.....	2 611	....	.....	2 666	....	.....	3 181	....	.....	2 416	....	.....	.....	....
	.....	(7)1 292	....	.....	2 042	....	.....	2 451	....	.....	2 348	....	.....	3 687	....	.....	.....	....
	.....	2 299	....	.....	2 472	....	.....	2 514	....	.....	2 833	....	.....	.....	....	.....	.....	....
	.....	2 486	....	.....	1 986	....	.....	2 139	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	2 139	....	.....	(8)1 820	....	.....	2 458	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	.....	2 583	....	.....	(3)2 625	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	.....	2 931	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
Average (2)..	1 784	2 204	336	2 851	2 536	652	.....	2 665	....	.....	2 764	....	.....	3 140	....	.....	.....	....

NOTES.

(1) = Tensile briquettes of mortar from the concrete.
(2) = Contain flat Conshohocken stone placed flat.
(3) = Flat Conshohocken stone placed vertically.
(4) = Very slow setting, partly fractured in removing from mould.
(5) = Very slow setting.
(6) = Cube spalled, area about 35 sq. in.
(7) = Corner broken, area about 138 sq. in.
(8) = Cube frozen on work.
(9) = Cube spalled, area about 136 sq. in.

TABLE 1.—Tests of Concrete Cubes from Walnut Lane Bridge.

Composition.	Age, 30 days.			Age, 60 days.			Age, 90 days.			Age, 120 days.			Age, 180 days.			Age, 1 year.		
	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)
	6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.	
1:2:2 (2)	2 462	2 111	341	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	...
1:2:5	1 841	2 055	197	4 236	.....	509	3 426	2 937	377	3 893	2 736	412	5 032	2 625	670	3 890	3 208	432
	2 089	.....	234	3 482	.....	442	4 684	2 875	....	3 702	.....	393	3 916	.....	438	3 664	2 694	435
	3 258	.....	349	4 009	.....	514	4 116	2 181	573	.....	.....	....	.....	.....	....	3 650	.....	209
	3 851	.....	597	4 452	.....	384	4 391	.....	504	.....	.....	....	.....	.....	....	3 001	.....	333
	.....	.....	....	3 150	.....	434	3 209	.....	478	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	3 164	.....	515	3 220	.....	489	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	2 993	.....	454	3 959	.....	515	.....	.....	....	.....	.....	....	.....	.....	....
Average	2 760	2 055	344	3 641	.....	465	3 858	2 664	489	3 768	2 736	403	4 474	2 625	554	3 526	2 951	375
1:2:5 (2)	2 372	2 111	406	(3)4 111	3 500	843	(3)4 655	2 694	681	.....	3 583	....	4 671	3 805	689	5 072	4 111	455
	2 010	2 861	496	3 512	.....	664	3 918	3 986	690	.....	.....	....	3 760	.....	659	4 375	4 264	477
	1 634	3 174	535	.....	.....	....	.....	4 330	....	.....	.....	....	4 201	.....	611	.....	.....	....
	1 948	.....	646	.....	.....	....	.....	2 806	....	.....	.....	....	.....	.....	...	.....	.....	....
	1 879	.....	369	.....	.....	....	.....	2 764	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 406	.....	375	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 395	.....	411	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 712	.....	451	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 689	.....	444	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
Average (2)	2 226	2 715	459	3 812	3 500	754	4 287	3 314	686	.....	3 583	....	4 211	3 805	653	4 724	4 188	466
1:3:6	1 091	1 440	385	2 696	2 033	291	3 568	2 097	676	3 613	1 833	238	2 683	3 479	333	3 676	.....	375
	1 993	(5)1 354	394	3 119	2 153	398	3 507	2 472	637	2 157	2 403	362	1 817	2 202	245	3 023	.....	444
	1 754	1 778	392	3 223	2 097	448	3 301	1 777	531	2 414	2 056	443	2 447	2 208	367	.....	.....	....
	2 018	1 916	218	2 704	1 896	387	2 888	1 743	526	2 118	2 861	225	2 453	2 833	273	.....	.....	....
	2 016	1 986	436	3 882	1 799	407	3 679	1 833	670	3 111	.....	339	2 168	2 708	421	.....	.....	....
	2 456	(9)1 694	539	4 017	1 903	481	2 536	2 861	469	2 962	.....	294	3 176	.....	393	.....	.....	....
	(4) 850	1 924	230	2 249	1 687	448	3 097	2 125	491	3 465	.....	457	2 908	.....	290	.....	.....	....
	1 678	1 888	363	3 151	2 083	400	2 829	1 666	554	4 326	.....	576	2 586	.....	236	.....	.....	....
	1 669	.....	253	3 215	1 861	603	2 234	1 916	358	4 256	.....	458	5 074	.....	493	.....	.....	....

It will be observed that:

(1.) The 12-in. cubes have a strength of about 80% of the 6-in. cubes;

(2.) The 12-in. cubes containing large stones embedded in them average from 25 to 35% stronger than plain cubes;

(3.) The strength of the concrete shows a fairly regular increase with age;

(4.) The tensile strength of the mortar taken from the concrete is from one-fifth to one-eighth of the compressive strength of the concrete;

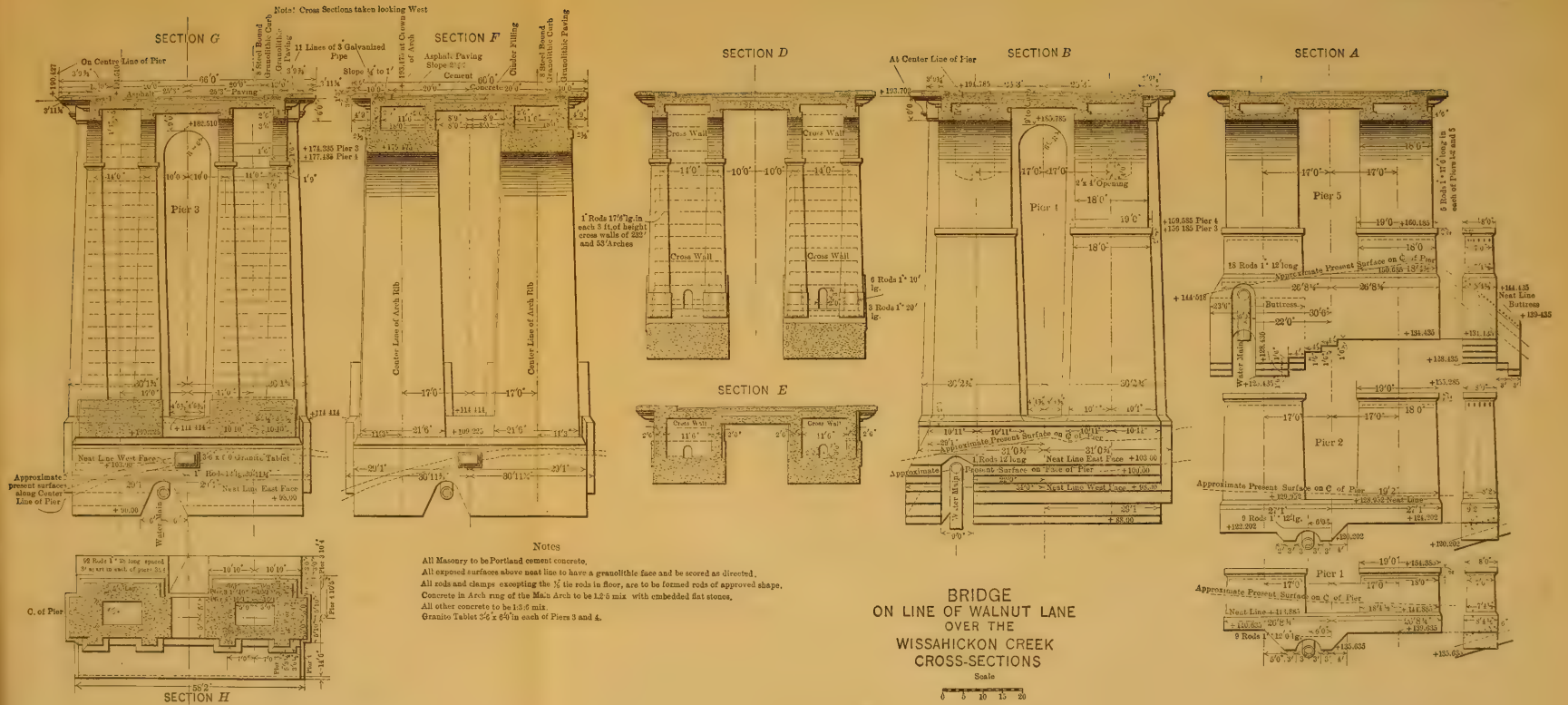
(5.) The strength of the 6-months old concrete may be assumed to be at least 2 000 lb. per sq. in.

The crushed stone used in the early part of the work and throughout the construction of the first main arch rib was a very hard black shale of high shearing strength and well graded in size, but was so sharply angular that it was gritty in the concrete and was thought to be less smooth working than limestone. Accordingly, a change was made to crushed limestone, with very satisfactory results. The composition and properties of these stones were as follows:

		Limestone.	Black shale.
Composition:	Passing No. 10 sieve	1.9%	1.5%
	" ¼-in. sieve	15.5%	12.5%
	" ½-" "	44.6%	34.7%
	" ¾-" "	77.6%	70.4%
	" 1-" "	96.8%	94.8%
	Retained on 1-" "	3.2%	5.2%
	Weight per cubic foot	98.9 lb.	90.9 lb.
	Specific gravity	2.84	2.66
	Percentage of voids	44.1	45.1

The embedded stone used throughout the work is known as "Conshohocken" stone—a laminated silicious rock of very high compressive strength—which comes from the quarry in flat shape and with tapering edges which facilitated the embedment of the stones in a radial direction in the arch rings, and permitted close packing with good bonding in the horizontal courses of the piers, etc.

The showing faces of the concrete throughout the bridge were made with a fine concrete or granolithic mixture, using for all except the balustrades a crushed black shale which had passed a ¾-in. screen and


Note: Cross Sections taken looking West
SECTION G
SECTION F
SECTION D
SECTION B
SECTION A
SECTION E
SECTION H
On Centre Line of Pier
At Center Line of Pier
Pier 3
Pier 4
Pier 5
Pier 2
Pier 1
Cross Wall
Center Line of Arch Rib
Neat Line West Face
Neat Line East Face
Approximate present surface along Center Line of Pier
Water Main
Buttress
Granolithic Paving
Asphalt Paving
Cement
Under Filling
1 Rods 17'6"lg. in each 3 ft. of height cross walls of 232' and 53' Arches
6 Rods 1" 10' lg.
3 Rods 1" 20' lg.
18 Rods 1" 12' long
9 Rods 1" 12'0"lg.
5 Rods 1" 8'0"long in each of Piers 1-2 and 5
Notes
All Masonry to be Portland cement concrete.
All exposed surfaces above neat line to have a granolithic face and be scored as directed.
All rods and clamps excepting the ¾" tie rods in floor, are to be formed rods of approved shape.
Concrete in Arch rng of the Main Arch to be 1:2:5 mix with embedded flat stones.
All other concrete to be 1:3:6 mix.
Granite Tablet 3'-6" x 6'-0" in each of Piers 3 and 4.
BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
CROSS-SECTIONS
Scale
0 5 10 15 20

was cleaned of dust. For the balustrades the crushed stone was granite, ¼-in. size. The mixture used was quite wet, and was deposited against the face forms by pouring from the spout of a coal scuttle, and the body concrete was laid against it in such a manner as to maintain always a coating of the mixture against the forms. As the mixture and the concrete were both of wet consistency, they readily united to form a monolithic mass. No case has yet been observed of the separation of the facing mixture from the body concrete. The face of the work throughout was finished by removing the forms as soon as practicable after the concrete had set and while it was still friable, and then with brushes and water washing off the surface sufficiently to expose the grit or aggregate of the facing mixture. The forms were generally removed on the day following that on which the concrete was deposited. In cold weather the forms were required to remain from two to three days to permit the concrete to harden sufficiently to maintain itself safely after the removal of the forms.

This process of finishing produces a surface which very closely resembles stone in texture, and, as it removes the cement film which segregates against the forms, all likelihood of surface cracking is eliminated. The tone or color of the surface, of course, depends on the aggregate used in the facing mortar. The black shale grit and the yellow bank sand used for facing the body of this bridge produce the general color and tone of Eastern granite. The mixture used in the construction of the balustrades, which constituted both the body and the face, was made with crushed granite screenings in place of the black shale grit used in the remainder of the work, and a pleasing contrast in tone was thereby obtained.

The use of this fine concrete for surfacing was for the twofold purpose of obtaining a more durable material, it being richer in cement, and a more uniform texture than would result from washing a concrete of coarser aggregate.

To facilitate the removal of the forms for finishing the surface, they were constructed with the uprights or studs set about 8 in. from the face of the work, and the planks of the forms, which were in width equal to the depth of the courses, were secured in place with easily removable cleats tacked to the uprights. On the top and bottom edges of the planks were fastened bead strips, 2½ in. wide, projecting

beyond the planks into the concrete to cause imprints. Courses were finished at the apex of the top bead, and, when the forms and planks were removed on the following day to permit the washing of the surface, they were raised one course, so that the bottom bead fitted into the half imprint left by the top bead. The work was thus built up course by course, in the smaller portions several courses being laid at one operation. No vertical beads were used, and no effort was made to imitate ashlar stone construction.

As the soffits of the arches could not be exposed for washing until the centers were removed, and the surface was then too hard to be removed by a brush, the impression of the lagging was removed by tool dressing. Pneumatic bush-hammers were used from a swinging scaffold, and the work was done before the floor was laid, so that the scaffold could be supported from out-looker timbers on top of the spandrel walls.

The construction of the main arch ribs was by blocks or voussoirs of a size limited to 75 cu. yd. each, always in pairs simultaneously—one on each side—so that the horizontal thrusting effect on the centering would be kept balanced.

The specifications prescribed that each section or block was to be constructed continuously from start to finish without cessation, in order to avoid joints, but in several cases it was found to be impossible to complete the section before dark, and forms were then erected to give to the day's work a radial face. Generally, it was found possible to complete a pair of blocks within daylight of one day—an output of about 150 cu. yd.

The number of large blocks was twenty-four—in twelve pairs—placed in the order of their numbers as shown by Fig. 3, with seventeen small blocks or keys. The first two cast were adjacent to the crown key, thus reversing the usual order of procedure in the construction of masonry arches, although it has been customary to weight down arch centers at the middle with a temporary load to prevent the rising at the crown caused by the thrust of the arch at the haunches. The arrangement was such as to leave seventeen small spaces for keying, the size of these spaces varying from 2 ft. 8 in. for the lower key to 2 ft. 0 in. for the crown key.

The method of construction used—building the voussoirs independently, scattered over the centering, to be joined later by the keys

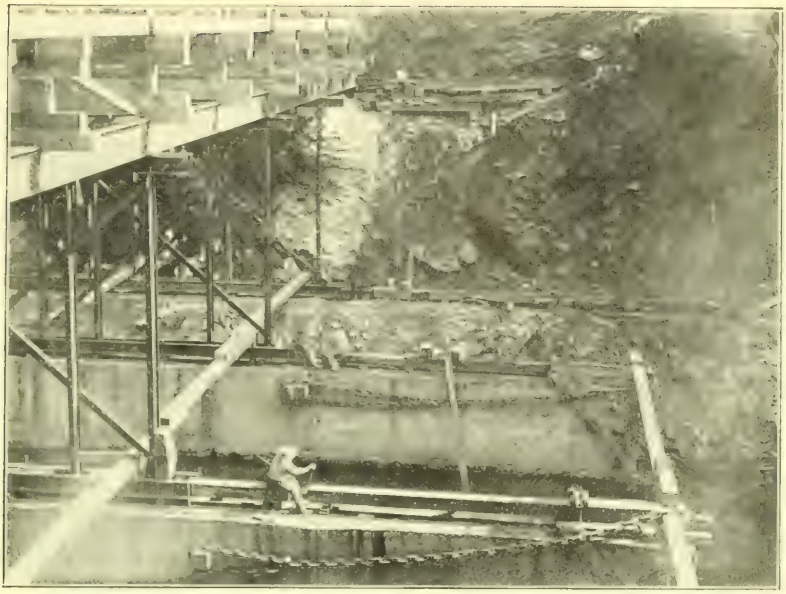


FIG. 1.—MOVING THE CENTERING, WALNUT LANE BRIDGE.

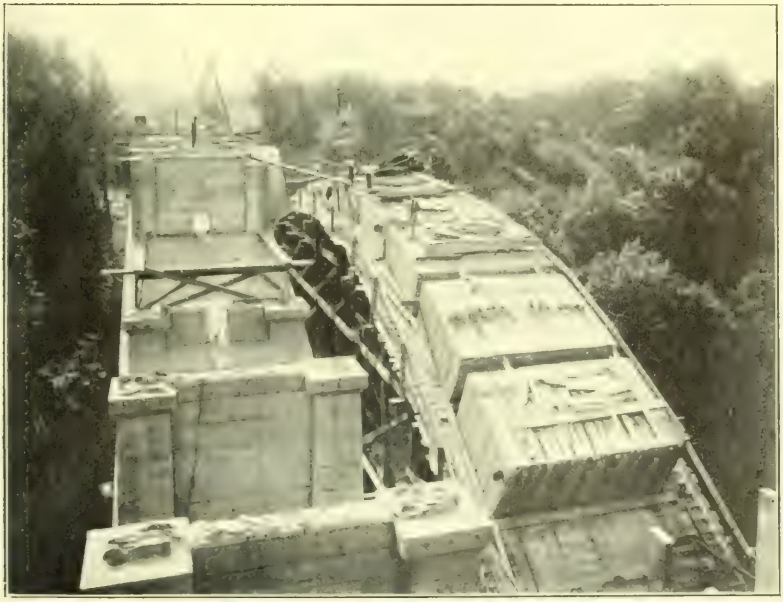


FIG. 2.—VOUSSOIRS OF NORTH RIB, WALNUT LANE BRIDGE.

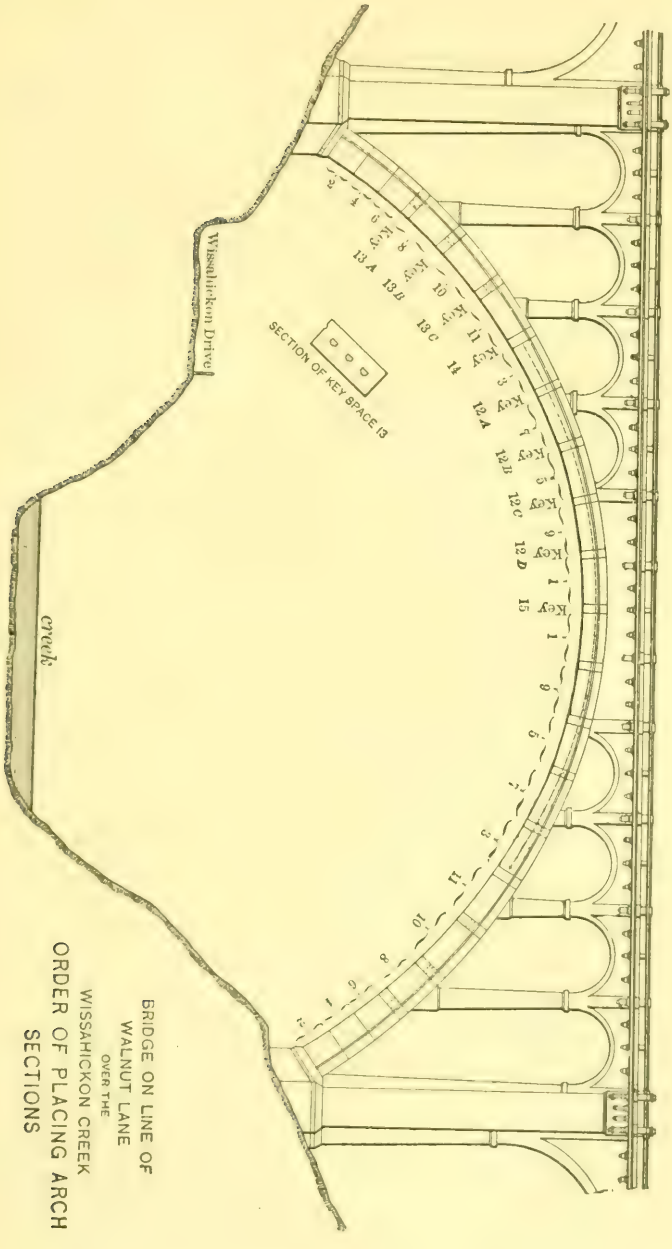
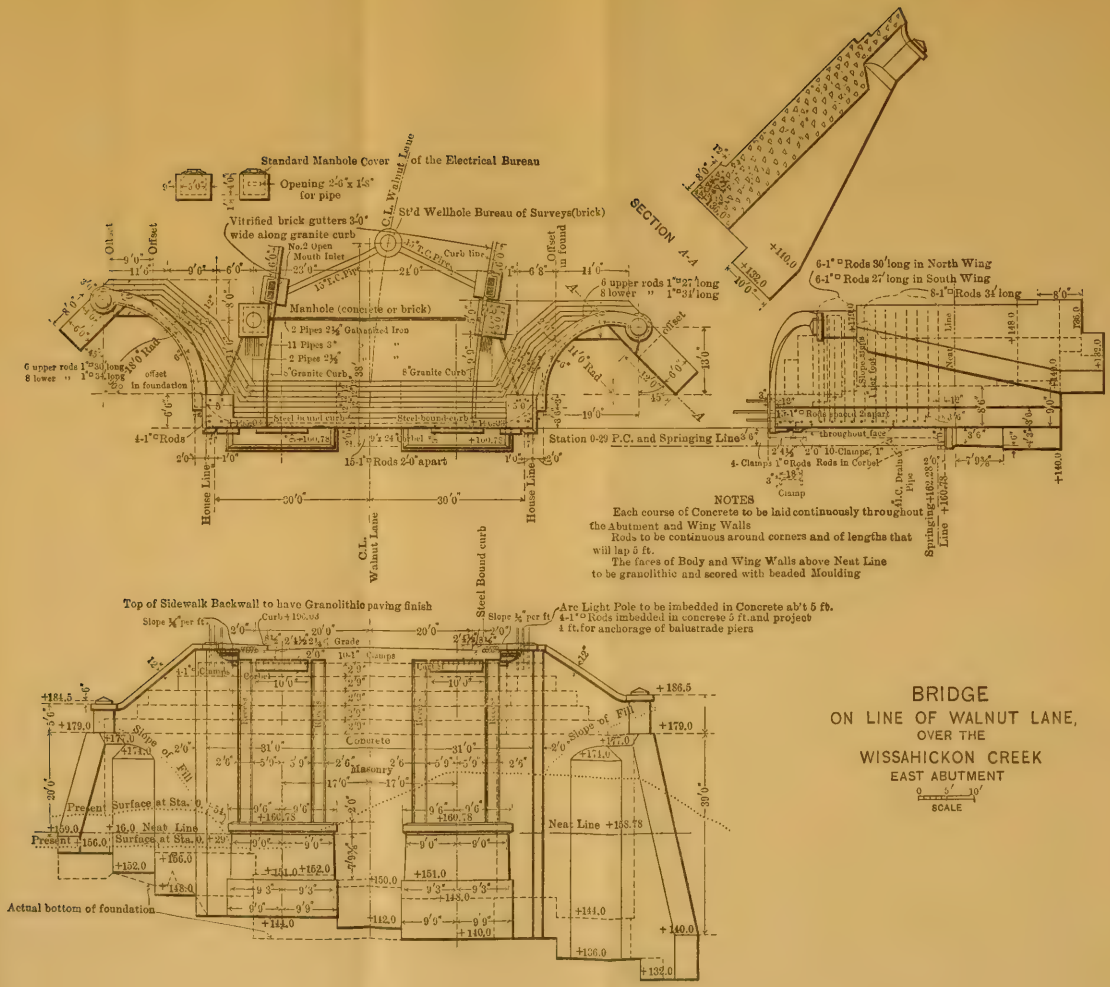

BRIDGE ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
ORDER OF PLACING ARCH
SECTIONS
Wissahickon Drive
creek
SECTION OF KEY SPACE 13

Fig. 8.

—required extreme care in the building of the forms, but the work was done very accurately, and all the edges of the ribs are in good line, and are entirely without kinks or waves.

The purpose of this method of construction was to distribute the load over the centering and permit the centering to attain its maximum deformation before laying, in order to prevent the occurrence of cracks in the arch, a condition which has generally been experienced in the construction of both stone and concrete arches under the usual method of beginning at the spring and proceeding continuously up to and keying at the crown. The utility of the method was demonstrated long before the completion of the first ring. It proved to be entirely successful, and although each main arch rib has in it forty joints, which are without sufficient adhesion of surfaces to give tensile value, there has not yet been discovered any disposition to open anywhere as the result either of loading or of temperature changes.

Some of the individual voussoir blocks on the steeper portions of the centering were required to be secured against sliding down hill on the lagging. This was accomplished in the case of the third pair of blocks—which were at the quarter points—by means of three 1¼-in., round, upset rods, made up to the required length with turnbuckles, and having 12 by 12 by ⅞-in. plate washers at the extreme ends. These rods pass over the crown at the mid-line of the arch, and Blocks 1, 3, 5, 7, and 9, were cast around them, the end washers being embedded in Block 3. No indication of movement of any of these blocks on the lagging could be found, and the one or two that became stressed must have been called into action by a slight movement of the joists. For the three lower key spaces on each side, the separation of the blocks was maintained by small concrete struts on the mid-line of the arch ring, three in each space—18 by 24 in. in the first, 15 by 20 in. in the second, and 12 by 18 in. in the third. As these struts must be built in with the key, they necessarily form an integral part of the arch, and as they must be subject to compressive stress in resisting the sliding of the sections, they must have, because of this initial stress, somewhat excessive pressure on them after the arch receives its full load. The load to be sustained by these struts was in a large measure dependent on the amount of friction on the lagging. Assuming that, if the blocks should move enough to break the bond of adhesion of the cement to the lagging, there would be no friction, the pressure on the struts


Standard Manhole Cover of the Electrical Bureau
Opening 2'-6" x 1'-3" for pipe
C.L. Walnut Lane
St'd Wellhole Bureau of Surveys(brick)
Vitrified brick gutters 3'-0" wide along granite curb
No.2 Open Mouth Inlet
Curb line
Offset
Offset in found
Manhole (concrete or brick)
2 Pipes 2½" Galvanized Iron
11 Pipes 3" "
2 Pipes 2½" "
Granite Curb
6 upper rods 1"□ 27' long
8 lower " 1"□ 31' long
6 upper rods 1"□ 30' long
8 lower " 1"□ 34' long
offset in foundation
4-1"□ Rods
15-1"□ Rods 2'-0" apart
Steel bound curb
House Line
30'-0"
C.L. Walnut Lane
Steel Bound curb
SECTION A-A
6-1"□ Rods 30' long in North Wing
6-1"□ Rods 27' long in South Wing
6-1"□ Rods 31' long
3-1"□ Rods (back) 2 apart
throughout face
Station 0-29 P.C. and Springing Line
2 Clamps 1"□ Rods in Corbel
Clamp
P.C. Prairie Pipe
Springing Line
NOTES
Each course of Concrete to be laid continuously throughout the Abutment and Wing Walls
Rods to be continuous around corners and of lengths that will lap 5 ft.
The faces of Body and Wing Walls above Neat Line to be granolithic and scored with beaded Moulding
Top of Sidewalk Backwall to have Granolithic paving finish
Slope ¼" per ft
Curb 4 196.08
Grade
Arc Light Pole to be imbedded in Concrete ab't 5 ft. 4-1"□ Rods imbedded in concrete 5 ft. and project 4 ft. for anchorage of balustrade piers
Steel Bound curb
Concrete
Masonry
Present Surface at Sta. 0
Neat Line
Slope of Fill
Actual bottom of foundation
BRIDGE
ON LINE OF WALNUT LANE,
OVER THE
WISSAHICKON CREEK
EAST ABUTMENT
SCALE

would be about 300 lb. per sq. in. of gross area. Any shrinkage of the concrete body of a key would entail the shortening of the struts by compression from arch stress before the main body of the key would be subjected to pressure. A shrinkage of as much as $\frac{1}{32}$ in. in the key would require for the corresponding compression of the struts a pressure of about 2 000 lb. per sq. in. if the modulus of elasticity of the concrete be 2 000 000 lb. per sq. in. This pressure on a small enclosed area would not cause any injurious effects on the material. The manner in which the keys were constructed, however, practically eliminated shrinkage, being in the nature of underpinning work. The mushy concrete was deposited in shallow layers, and more or less wedge-shaped stones were worked and rammed into it. The only visible effect of shrinkage in any of the keys was at the extreme top of the upper joint, where a very fine hair-line crack was distinguishable. In many not even this could be found because the concrete was banked and dammed up higher than the top edge, on the sink-head principle, this extra body being dressed off after hard set.

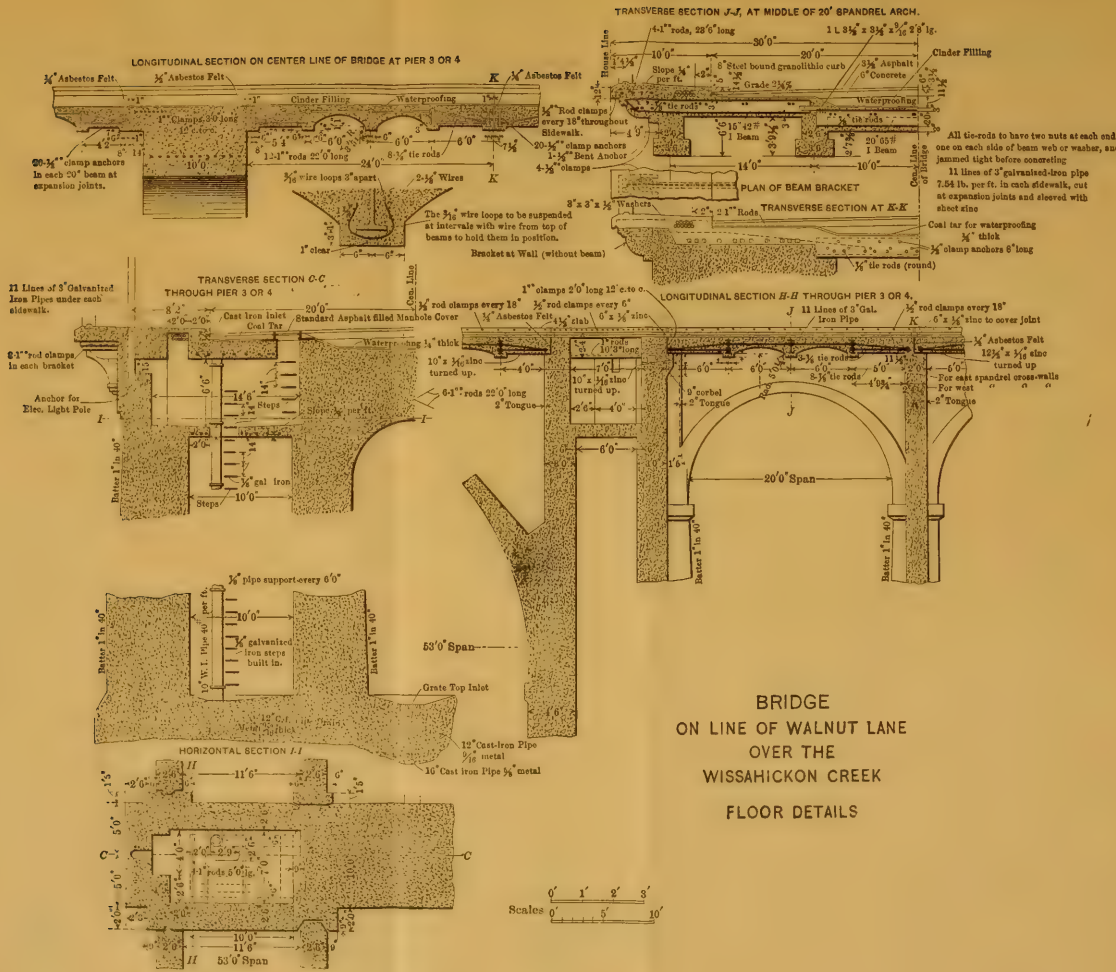
The anticipated hinge action of these struts developed as the load on the center increased, and it was demonstrated by their progressive cracking as the successive blocks were cast. The crack in each case opened on the upper or outer side of the struts, extending generally to within 6 in. of the under side, thus reducing the actual area for service to a fraction of what was provided, but it was evidently ample for the actual requirements. No special effort was made to grout up these openings, and any failure of the liquid of the concrete to penetrate and fill them would of course leave some small portion of the arch section unflushed and void, but the area thus lost was trifling, and, moreover, was at the least important part of the arch—the mid-line.

The theory of the construction of this arch, involving the principle of numerous keys and temporary hinges, logically called for a key space at the springing line, but it was omitted there because the satisfactory construction of a key would have been difficult, and the struts necessary for maintaining the space during the construction of the remainder of the arch would have been large enough to interfere somewhat with the proper deposit and compacting of the concrete of the body beneath the struts. The three lower blocks, therefore—Blocks 2, 4, and 6, Fig. 2—were cast in contact, and thus formed practically one section about 25 ft. long, directly on the skewback and

with its center of gravity about 4 ft. in front of the face of the abutment. The correctness of the theory referred to—that a key was necessary at the spring—was demonstrated when Block 8 was cast. The pressure on the falsework, at this stage of the construction, caused sufficient yielding of the centering to permit the heavy 25-ft. section to tilt forward and separate from the skewback at the outer side while bearing on the inner or lower edge. This action occurred at all four of the skewbacks, the cracks thus formed varying in size of opening from $\frac{1}{8}$ in. on one skewback to $\frac{3}{8}$ in. on another at the extrados of the arch, and extending to within about 12 in. of the intrados.

As the construction of the arch on the sections as tilted would have concentrated all the arch pressure on a fraction of the area of the ring at the skewback, it was necessary to push these sections back until their bearing was uniform over the whole width of the skewback. This was accomplished with screw-jacks and wedges acting under the joists and against the horizontal timbers of the falsework, and it is believed that, because of this adjustment, practically all initial stress in the arch ring was avoided. The distribution of the voussoir blocks over the centering, before constructing any of the keys, gave to the falsework almost all its load, and produced the maximum deformation in it before the keys closed the ring. The entire absence of openings in any of the joints between keys and sections, and of any cracks in the sections at any subsequent stage of the erection, is proof of the fact that the line of pressure was always within the middle third of the depth of the ring, for none of the joints have any tensile resistance to opening.

One of the ribs was subjected to an unintentional but interesting test of its stability by inadvertent unsymmetrical loading during the construction of the spandrel walls, which work was done after the centering had been removed. The work of construction over the arch was required to be carried on simultaneously on opposite sides, in order to avoid unsymmetrical loading, but, through a combination of circumstances, the rule was neglected for a part of a day, and a weight of about 100 tons of concrete was deposited in the spandrel walls of one section immediately over one of the quarters without any balancing load on the opposite quarter. As soon as the condition was discovered a careful examination was made of all the joints of the arch, but not the slightest evidence of distortion could be perceived. A pressure


LONGITUDINAL SECTION ON CENTER LINE OF BRIDGE AT PIER 3 OR 4
TRANSVERSE SECTION J-J, AT MIDDLE OF 20' SPANDREL ARCH.
TRANSVERSE SECTION C-C THROUGH PIER 3 OR 4
LONGITUDINAL SECTION H-H THROUGH PIER 3 OR 4.
PLAN OF BEAM BRACKET
TRANSVERSE SECTION AT K-K
HORIZONTAL SECTION I-I
Bracket at Wall (without beam)
20'0" Span
53'0" Span
Grate Top Inlet
Scales

BRIDGE
ON LINE OF WALNUT LANE OVER THE WISSAHICKON CREEK
FLOOR DETAILS

diagram, covering this condition, was made subsequently, and the center of pressure was found to be within the middle third of the arch ring. This is accounted for by the great weight of the arch ring itself.

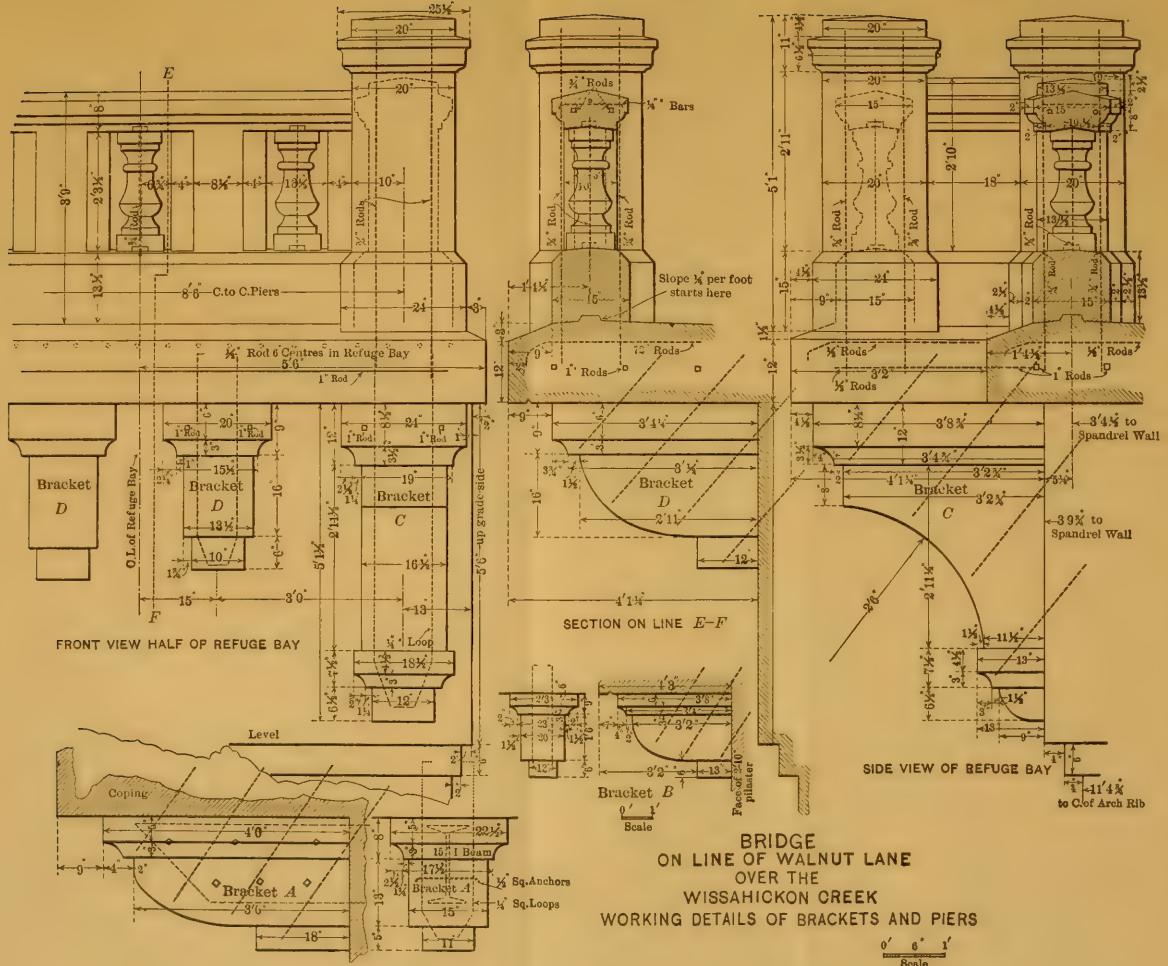
The centering was required to be maintained under the completed arch ring for at least one month. The first ring was keyed in a season of rising temperature (June 7th), and the wedges of the centering were struck one month later. The second arch was keyed in cool weather (October 17th), and the support was continued for 6 weeks. The stresses in the woodwork were high enough, apparently, to produce progressive deflection, so that, during the time of loading and subsequent seasoning, the falsework compressed sufficiently to lower the crown 3¾ in., this amount being the same for both ribs. In anticipation of this compression, the centering was originally set higher than the plan elevation. During the construction of the keys the woodwork was saturated with water as far as practicable by flowing water upon the bents through holes in the lagging, and the water was shut off as soon as the keys were completed, the purpose being to obtain the advantage of the shrinkage of the wood in drying out to withdraw gradually from the arch some of the support of the centering, and thereby facilitate the striking of the wedges. The utility of this process is doubtful when as high stresses are put in the wood as was done in this case, for it is probable that the softening of the unconfined fibers of the wood by the absorption of water did not operate to increase the height of the centering by the swelling of the wood, as was expected. The action of the wood during the period following the keying of the arch, apparently, was the continued yielding under the load, indicating that the elastic resistance was less than the imposed stress, and, as the deflection of the crown of each of the two main ribs was only ⅛ in. at the time of striking the wedges, it would seem that the elastic resistance of the whole structure of the centering was just sufficient to carry so much of the weight of the arch ring as would deflect the arch ⅛ in.

The centering was designed by the contractors. Its general arrangement and details, with stresses and sizes, are shown on Plate XLIII. All bents were supported on concrete piers built on the rock, with the exception of four intermediate ones, which were supported on 24-in. I-beam stringers spanning the creek bays. The lower story of the high bents was made of steel beams and channels braced with angles.

The remainder was of yellow pine timber. The working stress in the 10 by 10-in. posts, which had up to 13 diameters of unsupported length, was about 1 000 lb. per sq. in. As the sills, blocking, and wedges, were all 10 in. wide, the area of side-grain bearing was the same as the end grain, and the unit stress, therefore, was the same. The resulting crushing of the fibers of the yellow pine sills where they bore on the 10-in. oak wedges clearly showed that the stress was beyond the proper working load for side-grain bearing. The extreme fiber stress in bending on joists and on caps of bents was 1 200 lb. per sq. in., and ample strength and stiffness in these members was evident. The steel I-beam stringers were stressed at 16 000 lb. per sq. in., and the resulting deflection, therefore, was normal, and had no effect on the concrete work. The steel I-beam posts of the lower story had a stress of from 5 700 to 8 500 lb. per sq. in., according to the unsupported length between points of attachment of the diagonal bracing. The 6-in. steel rollers under the high bents were loaded, during the erection of the arch, to 3 400 lb. per lin. in., assuming that the rollers were all equally loaded, which is hardly possible. The total weight of the falsework on the rollers of the middle bent was 100 tons.

A peculiar feature of the structure was the method of connecting the curved joists. There were twenty-five lines of these, lapping at each bent and pinned together there with 3-in. round oak treenails. Instead of being notched to bear vertically on the caps of the bents, the joists were left straight and smooth on their underside, and beveled timbers on top of the caps of the bents were dressed to fit under them and receive their pressure. This construction produced horizontal as well as vertical forces to be resisted. In the lower bents the lateral thrust was greater than the vertical because of the steep inclination, and it was resisted by horizontal struts or rangers extending clear across. At the higher bents, where the inclination was flatter, the horizontal effect of the joist bearing was taken up by plank chocks spiked to the joists above and against the beveled timbers.

The designer assumed that the pinning of the joists would cause them to act as an arch and thus carry a considerable portion of the weight of the concrete ribs. It was apparent, however, that they did not act in this way, and also that the lagging did not arch. The length of arc was about 280 ft. and the shrinkage of the lagging in drying out was more than sufficient to accommodate the shortening



BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
WORKING DETAILS OF BRACKETS AND PIERS

0′ 6″ 1′
Scale

of the arc in the lowering of the crown by the compression of the centering, and, although the joists were very carefully bored for the treenail pins, the play in the holes and the compression of the fibers was evidently sufficient to prevent any observable arching. The placing of the joists was commenced at the crown and proceeded downward on both sides, the lower joists being fitted into the end spaces, the tendency of the joists to slip down being resisted by their linking over the crown, whereby the chain of joists was in tension before the lowering of the centering began. The proportion of rise to span is such that a change in length of arc produces almost exactly the same amount of change in the elevation of the crown. Therefore, for the deflection of $3\frac{3}{4}$ in. at the crown, the line of joists must have shortened $3\frac{3}{4}$ in., and, as the end bents depressed $\frac{1}{8}$ in. each, and as there were nineteen pin joints, the yield at each joint, by play and by compression of the wood, must have been $\frac{3}{16}$ in. When the centering was taken down, it was found that many of the treenails were permanently bent into a zigzag shape, presumably due to the effort of the linked joist to close on the pins.

It seems probable that a much stiffer and somewhat more economical structure would have resulted from fitting the ends of the joists and abutting them.

An unsatisfactory detail appeared, also, in the high blocking used between the first and second stories, and between the second and third stories. The caps and sills of the bents at these points were made double, to furnish stiffness between the posts for anticipated jacking, and 20 in. of space was then provided for the jacks. This space, therefore, had to be made up in blocking. Between the first and second stories, two 10 by 10 by 20-in. blocks were used, and between the second and third stories one 10 by 10-in. block and two oak wedges 10 in. wide and 5 in. thick at mid point. There was thus 60 in. of wood bearing across the fiber, made up of six pieces 10 in. wide. Its objectionable nature appeared in the unsymmetrical yielding of the wood under long-continued pressure, causing the pile of blocks to buckle and begin to topple. No actual failure occurred at any point, but it was evident that if the structure had been required to sustain its maximum load much longer some of the piles of blocks would have collapsed. The unsymmetrical yielding seemed to be due, at least in some cases, to an excess of sap wood on one side of the block—the

heart of the wood not being in the center of the stick. In some instances the sap wood split away from the heart wood, having swelled more than the heart wood, probably from absorption of water, and then buckled away.

It was observed that in the higher bents the caps became dished, on account of the greater compression of the interior posts. The concavity was as much as 1½ in., and the soffit of the arch rib at the crown, therefore, is necessarily that much curved, and the arch is that much thicker in the middle than at the faces. This dishing was not perceptible in the sills of the second story, and only slightly in the sills and caps between the second and top story, the transverse bracing probably acting to distribute much of the load to the outer posts.

The action of the falsework indicated that the unit stresses to which the wood was subjected, in side-grain bearing at least, were beyond its elastic resistance, and it is recommended, therefore, that the working stresses of wood in side-grain bearing, even for temporary duty, should be limited to 500 lb. per sq. in., and that the details should be made as compact as possible.

The means provided for adjusting the height of the bents, and for lowering them in striking the center, consisted of oak wedges, 10 in. wide and 20 in. long, tapering from 7 to 3 in. in thickness, placed crosswise of the sills in order to facilitate driving. When the center was lowered the wedges on the end bents at the skewbacks were loosened first by backing them out with sledges. At each of the four points they were found to have only a moderate pressure on them, proving that there was no considerable arch action on either the lagging or the joists. Then the wedges under the middle bent were slackened by backing them only enough to release a portion of the pressure, and successively all the others—working two gangs of men, one toward each end. A number of the wedges were so tightly held that they could not be started without having the weight on them partly taken up on jacks, one or two 15-ton ratchet screw-jacks being used for each tenacious wedge. One wedge refused to budge, even then, and it had to be split out with chisel bars. The evidently unequal loads carried by different posts of the same bent may be accounted for by different resisting values in the posts, and the consequent deflection and concentration of load by the caps of the bents. After all the wedges had been partially backed, the operation was repeated from the middle bent

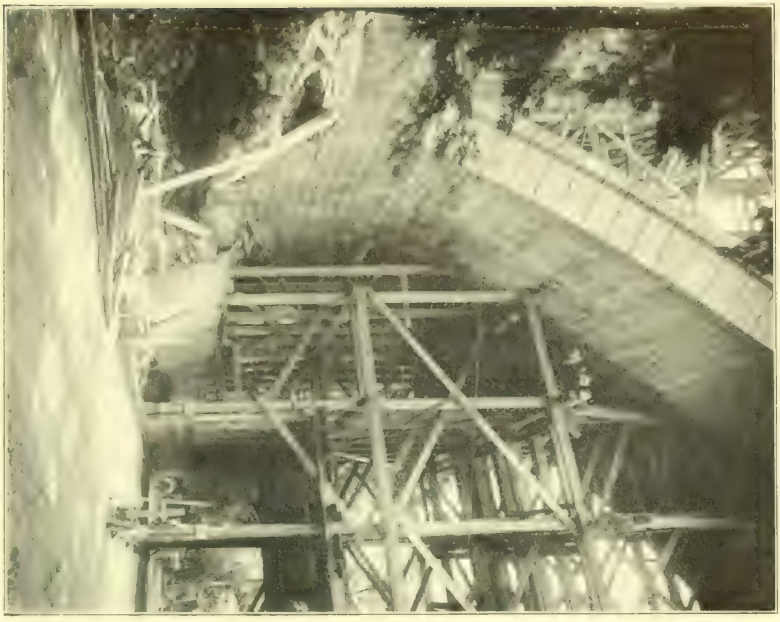


FIG. 1.—END OF CENTERING PARTLY MOVED, WALNUT LANE BRIDGE.

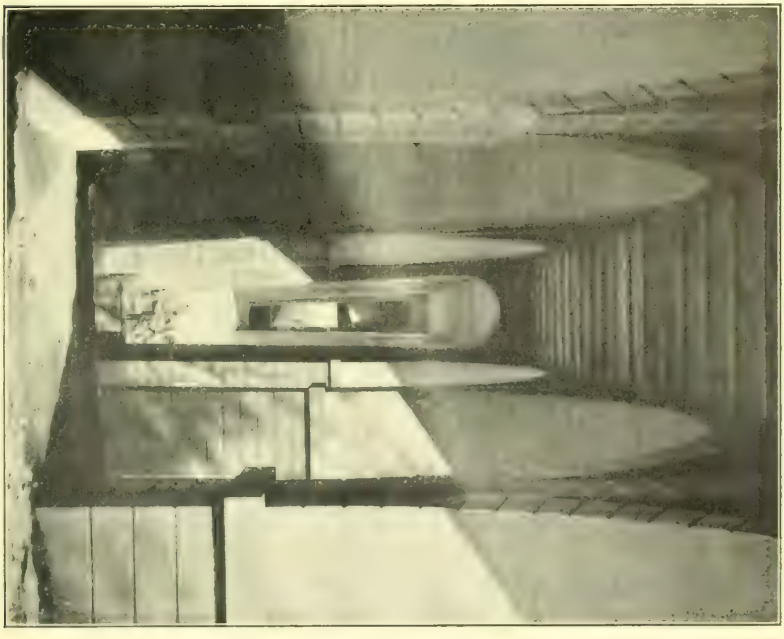


FIG. 2.—VIEW BETWEEN RIBS, WALNUT LANE BRIDGE.

to the ends, and all bents were lowered several inches clear of the arch. The longitudinal bracing of the bents acted as a truss, and carried the middle bents when their wedges were dropped, thus preventing any bents from lowering much below their immediate neighbors. Notwithstanding the apparently heavy pressure on some of the wedges, it was clear that, at the time of striking, the centering was carrying only a fraction of the weight of the concrete arch, for the deflection of the latter upon the withdrawal of the support of the centering amounted to only $\frac{1}{8}$ in. at the crown and $\frac{1}{16}$ in. at each quarter. This was the case in both ribs. The deflection was measured by three observers, each with an engineer's level, three fixed targets being used —one on the crown and one on each quarter.

One of the reasons for the economy found in the double-rib type of arch was the practicability of building one rib at a time, requiring only enough centering for one rib. Therefore the centering for the main arch was made for a single rib, while the falsework piers were made long enough for the two, and, after the first rib was swung, the whole center—with the exception of the end bents, which were moved independently—was moved over as a unit into position for the second rib. This was accomplished by using rollers under the bents, and jacks pushing on the sills (Fig. 1, Plate XXXVI). The steel bents had been erected on 6-in. solid steel rollers, and 3-in. wood rollers were inserted under the low bents. The jacks heeled against blocks chained to wooden spurs built into the piers for the purpose. The jacks were all actuated simultaneously, and a uniform speed of movement was maintained in order to prevent any skewing of the structure. This operation occupied less than three days.

The centering for the 53-ft. arches was an **A**-frame truss with a collar tie at mid height, but no tie at the heels; the thrust, therefore, was sustained by the piers. Computation of the dead weight resistance of the piers to overturning indicated that in two cases it was scarcely equal to the overturning moment, and that prudence would dictate the precaution of ties at the heels of the ribs. They were not used, however, and the construction was accomplished without any visible effect of thrust upon the piers. The dowelling of the courses of the concrete masonry of the piers, which was the result of finishing every course with the embedded stone projecting above the surface, is

doubtless to be credited with increasing the resisting moment of the piers by giving tensile value to the joints between the courses.

The vertical support of the trusses of the 53-ft. arch centering was obtained on the 6-in. projection of the coping, supplemented with a wooden trestle bent from the ground up in front of each of the low piers, and, in the case of the high piers, a timber corbel bolted to the face of the concrete. The bolts for the purpose of supporting the form timbers, wherever used throughout the work, were $\frac{3}{4}$ in. in diameter, and built into the concrete; and the nuts, 4 in. square and $\frac{5}{8}$ in. thick, were embedded from 6 to 8 in. As a rule, they were loosened slightly by unscrewing them one turn within a day or two after placing, in order to facilitate their ultimate removal, which was generally accomplished with little trouble. A few, however, were found to be gripped so tightly by the concrete that they were twisted off in the effort to unscrew them from the nuts, and had to be cut off with a chisel. This was done 1 in. or more below the surface, in order to permit a sufficient body of pointing to prevent the metal from rusting and staining the face. All holes left by these bolts were plugged with granolithic mortar and washed to match the face work.

The centers for the 20-ft. spandrel arches were built up and the lagging was nailed on before placing them in position, each center being handled and set as a unit. These were supported on the projecting caps of the piers.

The centering for the floor arches was suspended from the embedded steel beams with $\frac{3}{4}$-in. loop hangers or saddles over the tops of the beams, blocked with wood high enough to facilitate the removal of the block and the pulling out of the loop after the concrete had become self-sustaining. The centering was removed as early as was deemed safe, in order to get the hangers out with the least difficulty—under favorable conditions for setting, it was done within three days. A few of the hangers were held by the concrete so tightly that they could not be drawn, and were permitted to remain. The removal of the jack-arch centering was accomplished with a light scaffold suspended by single ropes through small holes cored in the floor concrete at suitable intervals. These holes, as well as those left by the withdrawal of the hangers, were not plugged, but were left open for the permanent drainage of any water which might get through the paving, but the spaces left by the removal of the wood blocks were filled with concrete.

FIG. 1. FLOOR SYSTEM, WALNUT LANE BRIDGE.

FIG. 2. DECK VIEW, WALNUT LANE BRIDGE.

All material was handled by a cableway of 800 ft. span, supplemented at the piers and on the floor by swinging boom derricks. The concrete mixing plant was under the cable.

The provisions for temperature changes throughout the structure have thus far proved to be effective in preventing cracking or spalling. The system adopted consisted in general of dividing the structure into sections, all the joints being locked with groove and tongue, and cushioned with a layer of soft asbestos felt. Steel reinforcement was not used anywhere to resist temperature effects. It has been observed in concrete structural work that unequal expansion of parts has caused rupture or spalling. Some parts—such as the rail of a parapet—may have less mass of body, and be also more completely exposed to the atmosphere, than other parts—such as the covered arch ring—therefore responding more quickly to changes in atmospheric temperature. They will then expand before the other parts, and something must yield. It has been found that, at points where such action is likely to develop, a sheet of soft mineral-wool felt, $\frac{1}{16}$ or $\frac{1}{8}$ in. thick, according to circumstances, will afford an elastic cushion which, by compressing, absorbs expansion, and, by swelling, follows contraction of the concrete, thus keeping out dirt and minimizing the appearance of the opening. Care must be taken, of course, to have the felt complete over the whole abutting surface of the joint. If any portion, particularly an edge of the joint surface, has the concrete in contact it may spall in expanding.

The floor structure, being of jack-arches, required staying at the joints, in order to take up the thrust of the arches. This was done with tie-rods. It required also double floor-beams at each joint in the middle aisle, independent of each other and separately encased in concrete. The joints are located so that in the side aisles they are over the cross-walls. Any movement in the floor structure at its joints necessarily involves a little sliding on bearings. Wherever this action is likely to occur, the seat of the sliding scale was made with a sheet of zinc, $\frac{1}{16}$ in. thick, flanged up at the back to prevent its dislodgment. Its smooth surface facilitates the sliding and it does not rust. The same device was used at the base of such of the balustrade posts or piers as occur at floor joints. The joint was necessarily made on the mid-line of the supporting bracket and wall, but it was not desirable to split the pier, so the half of the pier which had to be built over

the sliding side was bedded on sheet zinc, and the joint at the side of the pier in the balustrade completed the provision for movement.

The joints in spandrel walls were made about 24 ft. apart over the main arch—one alongside each cross-wall—and in the 53-ft. arches, because of the greater depth and consequent rigidity of the walls, the joints are only about 18 ft. apart. In the floor the joints are as much as 60 ft. apart, and the movement has not been enough at any one of them to develop a crack in the paving of the roadway, which is continuous over the joints. The joints in the sidewalks have the felt cushion.

Provision was also made for hinge action in the spandrel arches, due to the change of relative elevation of their springing points with the rise or fall of the crown of the main arch. The change of elevation of the crown from variation in temperature is $1\frac{1}{8}$ in. The spandrel piers, therefore, move up and down, each an amount proportionate to its distance from the middle of the span. Thus, the one nearest the end rises approximately one-fifth of $1\frac{1}{8}$ in., the next one two-fifths of $1\frac{1}{8}$ in., and the third three-fifths, etc. As this movement is necessarily vertical, the effect on the small arches is a bending with rotation about their springing points. To minimize the tendency of this action to spall off the concrete of the intrados at the spring, or to open cracks higher up, a sheet of asbestos felt was built in the whole width of the rib, 3 ft. 6 in., extending in from the soffit 9 in. Of course, this reduces the end bearing and increases the clear span of these arches, but they are of more than ample size for their duty.

Steel-rod reinforcement was used in general only at points where it was thought that the concrete was likely to crack on account of shrinkage or other indeterminate stresses. At the few points where load conditions produce tensile stresses, the amount of tension is so low that the flat embedded stone would furnish ample value to carry them, several tensile tests of the stone used showing strengths of from 1 000 to 1 200 lb. per sq. in. These points are the sidewalk brackets which are opposite cross-walls where steel beams were not used in the side aisles of the floor, and the 4 ft. 9-in. overhanging floor slab which is supported on the sidewalk brackets. In the former, two 1-in. square rods were embedded, extending into the concrete of the middle aisle for transverse tie purposes, and in the floor slab four 1-in. square rods were embedded in a longitudinal direction. In addition to these, trans-

FIG. 1.—BOTH RIBS, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

FIG. 2.—VIEW OF WALNUT LANE BRIDGE FROM THE CREEK

verse rods, ½ in. square and 18 in. apart, were embedded in the sidewalks to prevent the possible tendency to crack longitudinally at the point of overhang—the outside line of the spandrel wall. For similar reasons, rods were embedded transversely in the spandrel piers on the main arch, where the cross-web walls are much less in thickness than the pilasters which they connect; also, in the high hollow shafts of the main piers with sharp angles in section and overturning stresses, also in the abutment wings, and in the long and slender hand-rails of the balustrades.

The likelihood of concrete becoming detached from large flat surfaces of steel was provided against by the use of steel anchorage at the few points where such action would be possible. These were the projecting ends of the 15-in. floor-beams where they form sidewalk brackets, and the outer edge of the 20-in. beams at expansion joints, and the bottom flanges of all beams. The encasing concrete is clamped to the beam webs by ½-in. bent bars passing through holes in the webs, and the flange concrete is tied up with $\frac{3}{16}$-in. wire loops 3 in. apart, connected, and therefore spaced, with side wires welded to them. The concrete of the sidewalk brackets was supplied in addition with ¼-in. square steel loops tying it to the sidewalk slab. The aim throughout the design was to counteract every conceivable tendency of the concrete to open cracks from any cause, and the monumental character of the structure demanded that every precaution be taken to ensure the utmost quality of durability in every part of it.

It is probable that no completed structure of considerable size, simple or elaborate in design, has been regarded by even its author as perfect. Doubtless every progressive designer studies each structure, as it is being built and after it is finished, remembering his reasons for certain features and passing critical judgment upon their merit, and determining how defects may be corrected, or the cost reduced, or the appearance improved. As more is learned from failures than from successes, the most interesting comments on anything may be criticisms of it.

As architectural appearance—beauty of form and grace of proportion—is largely a matter of taste, the author of a work of art is hardly a competent critic of it, and, therefore, suggestions for improvement of the artistic quality of the design of the Walnut Lane Bridge must be left for others. Such modifications of structural details, for economi-

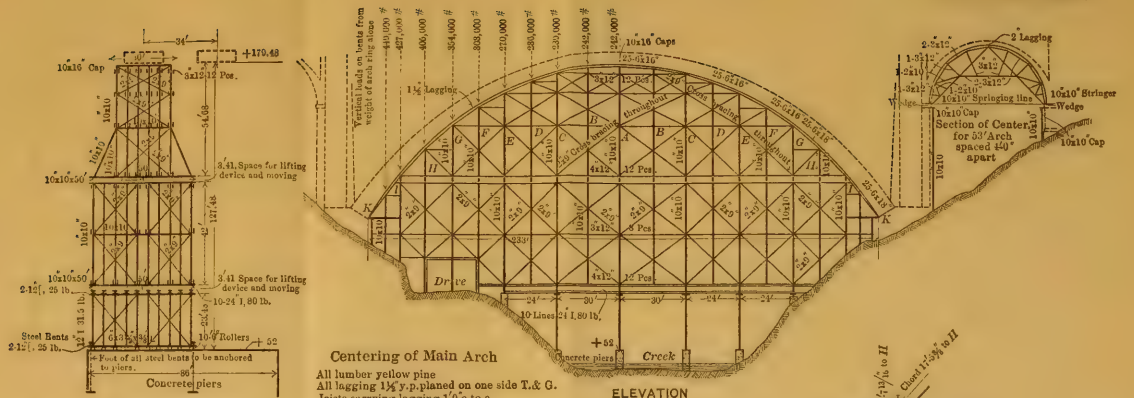
cal or other reasons, as have appeared desirable may be of interest to the Profession.

The most persistent enemy of masonry work—stone, brick, or concrete—is water when it finds entrance to it. The provision made in this bridge for drainage has not proved adequate for the complete exclusion of water from the joints, and this—a common failing in such structures—is apparently the only feature of this bridge which can be regarded as a structural defect. The points where it appears, however, are few in number, and the consequences are no more serious than slight blemishes in the appearance. Although the driveway is asphalted and has not a discernible crack in it, some water seems to get through it, and it finds outlet below where it shows on the nearest surface, and produces efflorescence. The joints in sidewalks and curbs are necessarily uncovered, and, consequently, some water finds its way through them, spreads along the floor seat ledges, and appears in stains on the finished faces of some of the abutments and piers. It is intended, in future work of similar character, to construct grooves in the seats and drain them into vertical conductors or down-spouts to be built into the wall joints and below them through the piers or abutments to some available point of discharge at or below the surface of the ground. In other bridges, efforts to prevent the infiltration of water into such joints, by the membrane method of covering the joint with web and elastic coating, have not been entirely successful, but it is believed that thorough drainage of the joints will prevent seepage to the front.

Some economy might have been attained, without lessening the stability or the durability of the work, by omitting the 18-in. concrete sheeting between the ribs of the 53-ft. arches—making four ribs, each 4 ft. wide, instead of two barrels, 18 ft. wide—and by making the spandrel walls 2 ft. thick instead of 2 ft. 6 in. Less steel reinforcement in the piers would have been justified with the high grade of concrete secured by the embedment of the flat stone, which by its complete bonding and great tensile strength resists shrinkage and other rupturing stress.

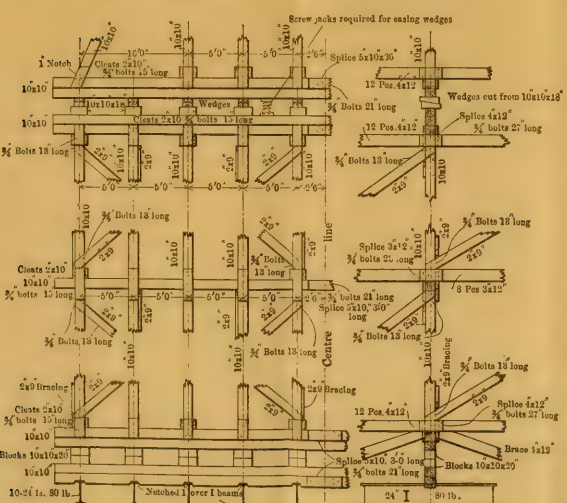
These are the only modifications of the design which have suggested themselves during the two years of construction of the bridge, which was a period of close observation and test of merit of the design.

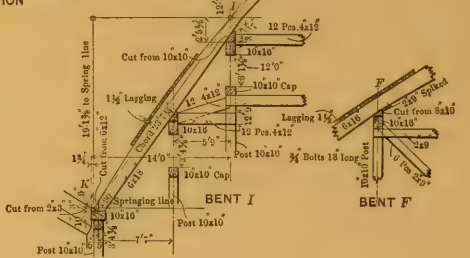
The class of concrete adopted and used, together with the character



Centering of Main Arch

All lumber yellow pine
All lagging 1½" y.p.planed on one side T.& G.
Joists carrying lagging 1'-0" c.to c.
Protection over drive 2" planks





BRIDGE
ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK
CENTERING

Scales. Elevation and Section. 0 5' 10' 20'
Details. 0 1' 5'

of the surface finish, and also the provision for expansion and contraction, represent the established practice of the City of Philadelphia in its bridge work, and, therefore, were not experimental. The method of constructing the main arch, as prescribed in detail in the specifications, was new, but the results fully justified it as scientific and practical, and it has been used since on similar though lighter work with equally satisfactory effect.

The total cost of the bridge, including electrical conduits and lamp standards, bush-hammering, and all extra work, was $267 000, which gives a rate of $7.60 per sq. ft. of floor surface and $0.088 per cu. ft. of space—area of profile by width of bridge.

The prices in the proposal and forming a part of the contract were as follows:

The work covered by the plans—lump sum	$253 550.91
Bush-hammering soffit of main arch ribs—lump sum	1 800.00
Additional bush-hammering—per square yard	0.16
Additional concrete in foundations—per cubic yard	7.50
Additional concrete above foundations—per cubic yard	9.00
Additional steel—per pound	0.04

The bronze lamp standards at the ends of the bridge, and the trimmings and lamps on the iron pipe poles in the refuge bays—contracted for separately—cost $7 982. These poles were made high enough and strong enough to carry diagonal catenaries for the support of overhead trolley wires in the event of street railways crossing the bridge in the future, the purpose being to avoid all use of poles along the curbs. The total cost of bush-hammering the soffits of the main arch, approach arches, and spandrel arches was $2 147.

The total quantity of concrete of all kinds in the bridge is 19 080 cu. yd., of which 2 300 cu. yd. are in foundations and 3 100 cu. yd. in the two ribs of the main arch. The structural steel in the floor amounts to 138 tons, and the steel reinforcing rods in foundations, wings, walls, and floor amount to 54 tons.

The centering of the main arch required 885 cu. yd. of concrete, 334 000 ft. B. M. of lumber, 116 tons of structural steel, and 16 tons of bolts, washers, and nails.

The work of construction extended over two years—from July, 1906, to July, 1908—concrete work having been suspended for three months during each of the two winters. The bridge was dedicated with appropriate public ceremonies and opened to travel on December 16th, 1908, after the grading of the Walnut Lane approaches was completed.

Honorable mention is due to the contractors, Messrs. Reilly and Riddle, for the creditable manner in which they executed the work, and for their resourcefulness and energy in its prosecution.

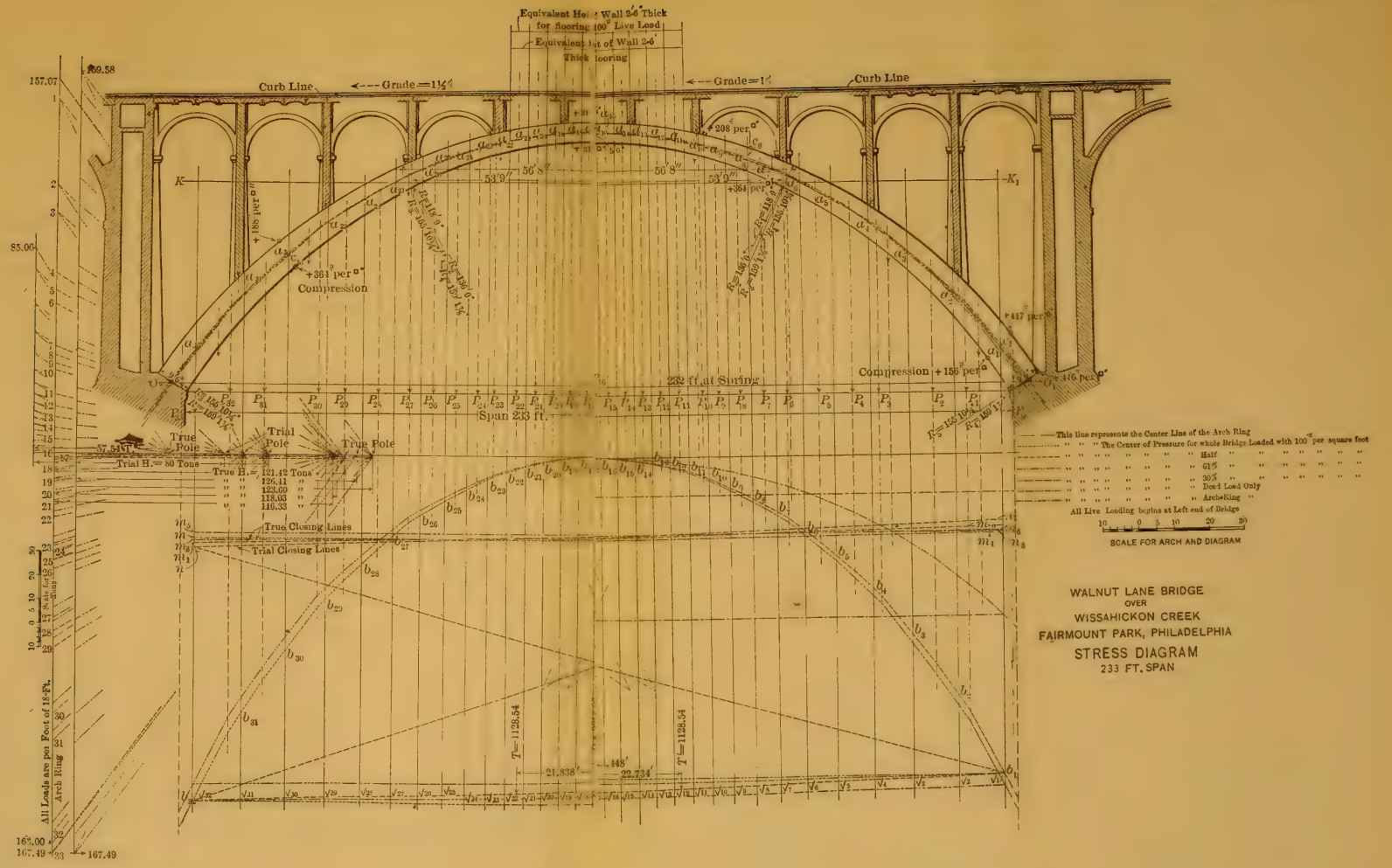
Extracts from Specifications.

The work covered by Item 1 of the proposal includes all the work necessary to make the bridge complete as per plans, and consists of excavation for foundations, grading at Piers 3 and 4, construction of all concrete and reinforced concrete work, with necessary centering and forms, floorwork, paving, iron pipe conduits for electrical wires, drainage appurtenances, manholes, electric light poles, and filling behind abutments as far as the lines shown on the general plan.

The work covered by Item 2 of the proposal may or may not be ordered, and consists of bush-hammering the under surface or soffit of the two ribs of the main arch, with pneumatic bush-hammers, sufficient to obliterate all impressions of the lagging.

In all foundations, stones of derrick size may be embedded, and smaller stones may be embedded with them, but they shall not be closer than 2 in. to any face, and they shall be used only by depositing concrete first and joggling the stones down into it. The embedded stones in the foundations of Piers 3 and 4 shall be of flat shape, and, as far as practicable, inclined so as to be normal to the direction of the resultant thrust on the pier.

The concrete of the main arch ring shall be composed of 1 part cement, 2 parts coarse sand or gravel, and 5 parts crushed stone, with embedded flat stones. The crushed stone shall be from $\frac{1}{4}$-in. to $\frac{3}{4}$-in. in size and thoroughly mixed. The embedded stones shall be not less than one-man size, and of such shape that their thickness will be approximately one-quarter of the average dimension, and shall be of quality and character equal to the best of Conshohocken stone, and be approved as such. They shall be placed in radial planes as close together as practicable, always edgewise to the centering, and embedded by first placing a sufficient depth of concrete and then working the stones into it. At the points of bearing of the spandrel piers and of the spandrel walls a number of these stones shall project above the arch ring to form a footing and bond. The concrete of the 20-ft. spandrel arches and the 53-ft. approach arches shall be of the same composition, with similar stones, of the same character, and of suitably proportioned size, similarly embedded.


Equivalent Height of Wall 2·6 Thick for flooring 100 Live Load
Equivalent Height of Wall 2·6 Thick for flooring
Curb Line
Grade = 1½%
Grade = 1%
Curb Line
157.07
169.58
85.06
K
K1
+364 per □
Compression
+447 per □
Compression +155 per □
+416 per □
232 ft. at Spring
Span 233 ft.
True Pole
Trial Pole
True Pole
Trial H. = 80 Tons
True H. = 121.42 Tons
" " 120.41 "
" " 123.60 "
" " 118.03 "
" " 116.33 "
True Closing Lines
Trial Closing Lines
T = 1128.54
T = 1128.54
21.838'
22.734'
448'
All Loads are per Foot of 18-Ft. Arch Ring
167.00
167.49
167.49
This line represents the Center Line of the Arch Ring
" " " The Center of Pressure for whole Bridge Loaded with 100 per square foot
" " " " " " " " " Half " " " " " "
" " " " " " " " " 61% " " " " " "
" " " " " " " " " 30% " " " " " "
" " " " " " " " " Dead Load Only
" " " " " " " " " Arche Ring "
All Live Loading begins at Left end of Bridge
10 0 5 10 20 30
SCALE FOR ARCH AND DIAGRAM

WALNUT LANE BRIDGE
OVER
WISSAHICKON CREEK
FAIRMOUNT PARK, PHILADELPHIA
STRESS DIAGRAM
233 FT. SPAN

All vertical walls of neat work shall have embedded in the concrete approved hard stones, not less than one-man size, so that they approach not closer than 2 in. to any surface, and so that every course or layer will have a number of the stones projecting up from it to dowel or bond with the succeeding course.

Piers 3 and 4 must be complete to and including the skewbacks at least one month before the concrete of the main arch is deposited.

Each main arch ring shall be constructed in transverse sections in pairs—opposite sides simultaneously—to ensure symmetrical loading on the centering. These sections shall be of such length that each pair can be constructed in daylight of one day. A detailed plan showing the sections and the required order of construction will be furnished by the City. The sections shall be separated by spaces of about 2 ft. 6 in., which will constitute the keys of the arch. The keys shall be constructed not earlier than two weeks after the last pair of sections has been placed. The lower three key spaces on each side shall have on the mid-line three concrete piers constructed at the same time as the sections, to maintain the separation of the sections and prevent possible sliding on the lagging. These piers will be surrounded and built in by the placing of the concrete of the key. The joints between keys and sections shall be locked by tongue and groove 1 in. deep.

The design of the centering to be used shall be submitted for approval before proceeding with its construction. The falsework for the arches shall rest on rock or on masonry piers built on the rock. The design shall be such as to avoid, as far as practicable, bearing across the fiber of the wood, confining the pressure as far as possible to the end fiber, in order to reduce to a minimum the yielding of the structure as it receives the load. Means shall be provided for lowering the supports evenly and gradually under absolute control.

Expansion joints in the walls and all joints in the balustrade shall be locked by tongue and groove 2 in. deep, and have $\frac{1}{8}$ in. thickness of asbestos felt separating the surfaces. The balustrade joints occurring at floor joints shall have $\frac{1}{4}$ in. of asbestos felt and have the locking tongue only slightly beveled to permit free movement.

At the expansion joints in the floor, where the movement involves sliding of the floor slabs on the walls, the bearing of the sliding portion shall consist of a strip of $\frac{1}{16}$ in. thick zinc, of the width of the bearing, flanged up 1 in. at the back edge.

At the expansion joints in the sidewalks the electrical conduit pipes shall be jointed by wrapping each with 4 by $\frac{1}{32}$-in. zinc to permit movement.

The connection between the 15-in. beams and the 20-in. beams of the floor must be made with tapered, turned bolts. The connecting angle is to be riveted to the web of the 15-in. beam with four $\frac{7}{8}$-in. rivets, and the holes in the flange of the 20-in. beam are to be drilled in position

and reamed tapered to fit the bolts, which shall be not less than $\frac{7}{8}$ in. in diameter at the point of shear.

There shall be embedded in the concrete around the bottom flange of each floor-beam $\frac{3}{16}$-in. wire loops 3 in. apart, connected on each side of the beam above the flange by two longitudinal wires $\frac{1}{8}$ in. in diameter, and 3 in. apart, to hold the loops in position. There shall be no longitudinal connection below the flange. The longitudinal wires shall be secured to the loops either by welding, as in the "Clinton Wire Cloth," or by tightly wrapping, as in the "Wight Lock Woven Steel Fabric."

The vertical drainage pipes from the grate top inlets shall be 10 in. in diameter, 40 lb. per foot, asphaltum-dipped, wrought-iron or steel pipe. The drainage pipes through the foundations of Piers 3 and 4 and underground shall be 12-in. and 16-in. cast-iron pipe. The latter shall be laid with 3 ft. of earth over it, and shall extend to the rocks at the water's edge of the creek.

The electric light poles shall be complete, except lamps and wires, which are not included in this contract. The poles and fittings shall be ornamental, and the design shall be submitted and be subject to approval before erection. They shall be of such height that the lamps will be 21 ft. above the sidewalks. They shall be anchored and built into the concrete work as it proceeds. The poles over Piers 3 and 4 shall be of sufficient strength to serve as trolley poles.

The steel edging for the granolithic curb shall be furnished in lengths to match the joints in the sidewalks.

The whole surface of the floor concrete between the curbs, and also the buried portion of the face of the curbs, shall be thoroughly coated with No. 4 coal-tar distillate to a least thickness of $\frac{1}{4}$ in.

Bush-hammering of surfaces, if required to be done, shall be with pneumatic bush-hammers, and shall be sufficient to remove the surface film, exposing the grit, and to remove the impressions of joints of the lagging.

The contractor shall furnish, at points to be designated, in the arch centering, wooden tell-tale pendants with corresponding stakes in the ground to facilitate accurate observation of deflection as the work proceeds.

Where anchorage in the concrete is required to maintain the forms in true position, it shall be designed so that any projecting metal work can be removed to a depth of 3 in. from the face without damaging the face, and the cavities thus created shall be filled with granolithic mixture.

The grit in all granolithic face work, except the balustrade, shall be of dark hard stone from an approved quarry, and that in the balustrade shall be of crushed granite.

FIG. 1.—WEST APPROACH SPANS, WALNUT LANE BRIDGE.

FIG. 2.—EAST APPROACH SPANS, WALNUT LANE BRIDGE.

The cinder filling in the middle section of floor shall be selected material, and shall be thoroughly compacted by wetting and tamping.

To economize in centering and forms, the bridge superstructure may be constructed in halves—one side throughout the whole length at a time—but the two abutments and the foundations of all piers must each be monolithic.

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PAPERS AND DISCUSSIONS

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A REVIEW OF CHICAGO PAVING PRACTICE.

By P. E. Green, Assoc. M. Am. Soc. C. E.

To be Presented October 6th, 1909.

The matter of pavements in Chicago is in some respects a recent one, as compared with some eastern cities. Articles on New York 100 years ago, state that that city was tolerably well paved; probably Philadelphia and Boston were also; certainly, Albany was, at the time of Fulton's first trip with his steamboat. Sixty years ago, however, Chicago had no pavements except plank and graveled roads, though in 1860 it was a city of 100 000 inhabitants. The condition of the streets can easily be imagined. There are photographs in existence showing that evidently every lot owner established his own grade for his sidewalk; and Clark St. had an appearance which can only be matched in the smallest towns of the West. In 1855 two or three squares were paved with limestone blocks, and at about the same time some cobblestone pavement was laid.

The first real pavement was the Nicolson block, a pine block similar to the present creosoted block, but untreated. This was laid on Fifth Ave., between Lake St. and South Water St., in 1856. The blocks were laid with 1-in. joints filled with gravel and coal-tar, the foundation being 2-in. plank on sand. The life of this

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pavement was short, and it did not stay in favor very long, though a considerable yardage of it was laid before the Chicago fire in 1871. Up to 1865 about 25 miles of pavements had been laid, mostly pine block and gravel macadam.

After the fire, the grade of the whole down-town and near residence districts was raised by from 2 to 11 ft., the débris of the fire being used to make the filling. The writer, when making excavations for sewers and water pipes, during the last two or three years, has frequently encountered these old Nicolson blocks, still in a good state of preservation, after being buried in ashes, brick, etc., for nearly forty years.

It must be remembered that, until the grade was raised, a large part of the city was practically a swamp. In the Forties and Fifties the citizens frequently discussed the desirability of stone block pavements such as used in the East. It was the opinion then that it would not pay to spend much money on a pavement which was sure to sink out of sight in a year or two. It was this condition which brought about the use of plank roads. They were very serviceable for many years, and were active competitors of the steam railways of those days. The writer has read an article, written for one of the Chicago papers during the Fifties, which vigorously assailed the policy of abandoning the building of these roads, and went on to prove by quotation of freight rates that freight could be moved much more cheaply by team on such roads than by railway; that passengers could be transported just as rapidly and with less danger; and that, if the city were to spend on these plank roads the amount of money it proposed to give a certain railway, dividends of 30% could be earned.

In 1875 the city began to lay cylindrical cedar blocks, and for nearly twenty years this pavement was used more than any other. The blocks were 6 in. in depth, and, like the Nicolson blocks, were generally laid on a plank foundation, though a few streets had a macadam base. A great deal of this pavement still remains, but it is in very bad condition. When first laid, it had a nice, even surface, which was very attractive, and hundreds of miles of it were put down. About 1890 it began to fall into disfavor, and from that time its use declined, as its defects became glaringly apparent. Its first cost was one thing which made it popular, the average price per square yard being from $0.85 to $1.25.

In 1882 the first asphalt was laid, and during the same year the first granite blocks. The cost of these pavements was very high, and the use of asphalt spread slowly, but, from absolute necessity, the use of granite blocks spread very rapidly, and much of this pavement, laid from 1882 to 1886, is still in service. No care was taken to dress the blocks; they were only roughly squared, and were laid with wide joints. On account of these defects, the pavement was very noisy, but it was lasting. Brick was first laid about 1891. A brick pavement on Lake Ave., from 35th to 37th Sts., was laid in 1893, and is still in use, though badly worn. The bricks were not "repressed," but were vitrified. The foundation consisted of a course of brick laid flat. This pavement has given good satisfaction under heavy traffic.

One of the drawbacks to the rapid construction of good pavements in Chicago is the law under which such improvements are made, taken in connection with the financial condition of the city. Under the Illinois law, payment is made on the basis of the benefit derived by the adjoining property. In other words, whether or not a street has ever been paved, the adjoining property owners must pay for the entire improvement, unless the County Court, which confirms the assessments for the improvement, assesses a general benefit against the city. As the city has little money, and the bonding power is very narrowly limited, the policy is for the city to have the ordinance rescinded and the improvement abandoned if the assessment against it is more than it can pay, and so it works out that the adjoining property owners pay for at least 90% of all pavements laid in Chicago. The city does not pay for intersections. The various street railway companies pave their own right of way, a strip 16 ft. wide for double track. The whole arrangement is unjust, is a hardship on property owners, and should be changed. In an indirect way, it often injures property, as streets which should be repaved are allowed to remain for years, because property owners do not feel that they will be benefited by the improvement.

All work in Chicago which is to be paid for by direct assessment against abutting property owners is under the supervision of the Board of Local Improvements, composed of five citizens appointed by the Mayor. As the work done by this Board costs several million dollars every year, a considerable engineering force is required; but, as the finances of the city are in bad shape, rigid economy is necessary all

the time. Hence, under the present system, the engineering force is totally inadequate to do the work as it should be done. As in other public corporations, there is much duplication of work in the different departments of the Board of Local Improvements as now constituted. Undoubtedly, if this organization were to be remodeled, the number of men available in the Engineering Bureau would not have to be increased in order to do the work in a much more satisfactory way, but it is difficult to make changes in the method of doing public work. This statement is given as a reason for the methods which the engineers have adopted in order to do a great deal of work with a very small force. These methods are far from ideal, but the writer thinks that they obtain the best results, under the unfavorable circumstances.

In an ordinarily busy year, some three hundred streets and alleys are improved. As it is not possible for the small force of engineers assigned to this work to set stakes constantly for the curbing, grading, concreting, etc., much of this work must be done by the inspectors, who are not engineers. Line and grade are marked on permanent objects along the street, such as sidewalks, electric light poles, etc., and from these the inspectors and foremen must work. Ordinarily, some member of the engineering force visits each job every day, but he can rarely give more than an hour of his time to each piece of work, unless some particular point requires his attention.

In order that the inspectors and foremen may follow the engineer's instructions, each is furnished by the division engineer with what is called a "profile." This is not a profile, in the ordinary sense of the word, but is a plan of the work, with many explanatory notes. These notes show where the line is marked, the offset, the height of the grade marks from the top of the curb, the width of the roadway, the roadways of intersecting streets, the depth of the gutter below the curb at summits and inlets, the location of new catch-basins, etc. The inspectors and foremen are taught how to stake out the ordinary crown with Tees, after the curb is set. The curb, of course, is always put in before the concrete foundation, and hence is a guide to the men.

Of course, careless inspectors and foremen make mistakes occasionally, and there is trouble when the engineer makes his examination, but, as a whole, the system has worked better than could have been expected. Inlets and summits are sometimes reversed, crowns are too high or too flat, but nothing better can be expected until the

engineers in charge are given more men. At present an engineering force of about twenty men, with a salary list of about $33 000, handles $4 000 000 worth of work per year, scattered over the entire city. Fig. 1 is the "Profile" of a street paved by the writer in 1908. It will be noticed that this is not a detailed, accurate plan, but is really a memorandum for the guidance of the inspector and foreman.

Before discussing the different modern types of street pavements, various features, common to all in a greater or less degree, will be taken up. These may be classified under the headings: Width of Roadway; Crown or Form of Surface of the Roadway; and Drainage.

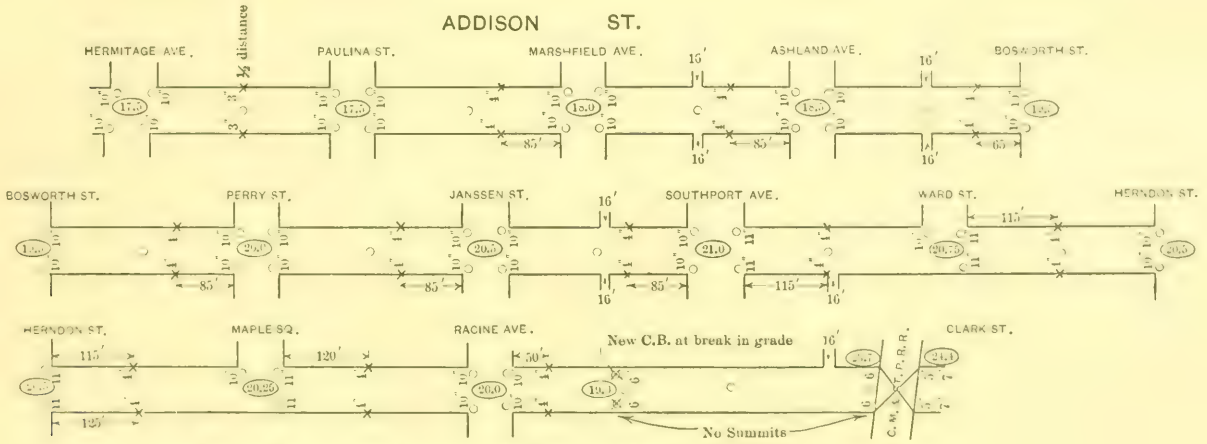
Width of Roadway.

The width of roadway is a question which causes the engineer more grief than almost any other. Property owners seem to have a general desire for a wide roadway. Many do not know, or more probably do not think, that the wider the roadway the more it will cost. Property owners, after receiving an assessment notice showing that the roadway will be reduced from the original width, have protested strongly, seeming to think that they were being defrauded of part of their rights, but when it was explained to them that their assessment would be much larger if the roadway were wider, they have been satisfied.

Chicago has a general ordinance for the width of roadways on streets of standard width. This, however, governs only the intersections of streets which have never been paved. When a street is to be paved, the ordinance ordering the improvement states the width of the roadway. The general ordinance specifies the following widths:

Width of street, in feet.	Width of sidewalks, in feet.	Width of roadway, in feet.
100	20	60
80	16	48
66	14	38
60	12	32
50	10	30
40	6	28
30	4	22

Probably any practicing engineer will agree that, for the average street, these widths are too great, and especially for very wide streets. Many residence streets have a width of 100 ft. between property lines,



Profile of Addison St., from W. line of Clark St. to W. line of Hermitage Ave.

Roadway 41 ft. Grade marks 2 ft. High. Crown 7 and 10.

Line marked on south walk 13 ft. offset (from face of curb)

Intersecting Rdys. Racine Rdy. 38′ offset North 17.5′ South 14.0′ Maple Sq. Rdy. — offset See Engineer.
Herndon " " See Engineer Ward " 38′ " 14′
Southport " 18′ offset 16′ Janssen " 34′ " 16′.
Perry " 34 " 16′ Bosworth " 34′ " 16′.
Ashland " 38 " 14′ Marshfield " " See Engineer.
Hermitage " " See Engineer.

× New Catch Basins.
· Old Catch Basins.
× Summits.
Grades.

FIG. 1.

and on such streets a 60-ft. pavement is the height of absurdity; it is expensive to lay and maintain, and certainly cannot vie in attractiveness with, say, a roadway of 38 ft., a grass space of 24 ft., and a sidewalk space of 6 ft., leaving 1 ft. between the walk and the property lines.

This tendency to make very wide roadways is more pronounced in small towns, which have not had many paved streets, than in the larger cities, which have had more experience. A certain town, having 3 000 inhabitants, proposed to pave its main street with a 60-ft. roadway, and then changed it to a 66-ft. roadway. An assessment for such a width nearly swamps the adjoining property owners. In another city, of 25 000 inhabitants, the main street has a paved roadway about 80 ft. wide; it looks like a deserted street on the busiest day, and actually gives the stranger a bad idea of the town.

If a street is wholly residential, and say 66 ft. wide, which is almost standard, a 30-ft. roadway is ample. This will allow two teams opposite each other to be backed up to the curb, and give a good passageway between, if the wagons are properly cranked. At street intersections, there is also ample room for vehicles of the fire department to swing around the corner. This is a vital point, and is sometimes overlooked by engineers, who go to the other extreme and advocate roadways which are too narrow. A long ladder-truck will have considerable trouble and delay in turning a corner on a narrow intersection, say 20 ft. There are generally electric light poles, lamp posts, etc., which may catch the ends of the ladders as they swing around. This objection, of course, can be largely overcome by building a wide sweeping intersection, but even that is not the only consideration. A very narrow roadway sometimes assumes too much of the appearance of an alley.

For an ordinary business street, not having car tracks, 38 ft. is a good width. A great many business streets in Chicago are of that width, and it is found to be ample.

If a street has a double line of car tracks, a wider roadway is necessary. The writer would then suggest a minimum width of 42 ft., but 48 ft. would be better. The reasons for this are obvious when it is considered that the ordinary street-railway right of way is 16 ft.

A congested district, such as "The Loop," in Chicago, furnishes a problem all by itself. In such a case the roadway can hardly be too

wide. But the sidewalk space is also congested, and must be taken into consideration. The roadways in "The Loop" are generally 48 ft. wide; part of State St. is 60 ft. wide. These are too narrow, but so are the sidewalks, and it is not possible to make much change.

Crown, or Form, of Surface of Roadway.

Apparently, there is little room for variation in the crown of the roadway. What is wanted is a shape which will drain rapidly and easily, and at the same time be as nearly flat as possible in order that it will not interfere with the freighting of heavy loads. For this purpose it is natural to use the circle or some similar curve. Many quite elaborate formulas have been suggested by various engineers as a solution of this problem. These generally take into consideration the width of the roadway, the grade of the street, and the desirability of having the maximum fall near the gutter. What most of them do not take into consideration is the fact that, in a flat territory like Chicago (and in some parts of nearly every city), it is absolutely necessary to form an artificial grade in the gutter. If the minimum depth below the top of the curb is 4 in., and the maximum is 10 in., it is obvious that a surface curve which depends entirely on the width of the roadway for its form, and, at the same time, establishes the center of the roadway at any fixed elevation, will not do. In the Chicago ordinances the form of the roadway is generally the arc of a circle. In construction work, however, it has been found by experience that this does not give the best results, and the parabola is almost always used. The formula is $y = \frac{b\,x^2}{a^2}$, in which b is the depth of the gutter below the grade of the center of the roadway, a is the half roadway, x is the horizontal distance from the center of the roadway, and y is the vertical distance below the grade.

This gives a crown which is not as flat at the center, and, at the same time, drops rapidly to the gutter in the last quarter; but, for most ordinary roadways, there is no practical difference between the two curves. It is also very simple to stake it out, and is easily taught to foremen, which is extremely important. The necessary stakes for the crown are generally set at the center and quarter points. In this curve the quarter points are always one-fourth of the total drop. This is very easily remembered, and the foremen are soon able to set the stakes accurately with wooden Tees. For very wide roadways

of, say, 100 ft., it causes too rapid a drop toward the gutter, and a curve intermediate between the parabola and a straight line must be used.

In regard to a crown of this form, however, it should be stated that it is intended for pavements having a rigid wearing surface which does not grind into dust. In other words, it does not apply to ordinary macadam, because it gives too flat a center, forming a perfectly flat, or even a concave, surface which is impossible to drain. The writer considers that the nearer the section of a macadam roadway conforms to straight lines, dropping both ways from the center, the better the surface is likely to wear.

One of the important points to decide is how high the crown should be, or, in other words, how much the pavement should drop from the center to the gutter. It sometimes happens that a combination of circumstances, such as established curb grades and street railway tracks, necessitates a very low crown. The writer has known cases, where, on account of these conditions, combined with the requirements of drainage, the drop was less than 2 in. from the rails to the gutter. This was at a summit in the gutter. This is bad practice, however, and should be avoided when possible. For ordinary purposes for smooth pavements, on streets having a roadway of from 28 to 38 ft., 6 in. should be the minimum, and, for a macadam pavement, 8 in.; on wider streets, the drop must be greater.

The late Andrew Rosewater, M. Am. Soc. C. E., when City Engineer of Omaha, used, for the height of the crown, the following formula, which has met with a great deal of favor:

$$H = \frac{W(100-4P)}{5\,000},$$

in which W is the width of the roadway, and P is the percentage of grade.

This is a good rule for park roads or boulevards, or for any street having a stiff grade, when little crown is necessary, but it makes the crown too flat for level, or nearly level, streets, where drainage has to be secured by artificial grades, and where the streets are not constantly cleaned. In general, however, it is very good. It is practically the same as the 2% rule mentioned below.

One of the best general rules known to the writer is that used by the engineer of the West Park Board of Chicago, who makes the crown

2% of the roadway. This works very well on residence streets, but not at all well where there are street-railway tracks and the grade is nearly flat, as the tracks cannot follow the changes of the gutter grade, and these control the crown grade.

The Drainage.

The third general point to be considered—the drainage—is probably the most important. The whole life of practically every pavement depends largely on this. All pavements in Chicago are designed to take care of the drainage, and if any pavement is defective in that regard it will be a failure, even if it can stand the heaviest traffic. When the writer speaks of drainage, he does not mean entirely the sub-surface drainage, though that is very important. In a large city, as a general rule, there is not much trouble with the sub-drainage, because the city is thoroughly sewered, and the opening of cellars and other excavations has drained the surface and lowered the general ground-water level. Of course, a naturally sandy soil needs little drainage, but a clay soil must be watched. If it is normally in a wet condition, some provision for its drainage should be made, such as laying tile pipe across the roadway and connecting them to the sewer inlets. In this paper, however, the writer will consider only the surface drainage.

If the surface of the town is rolling, and naturally drains well, there will be little difficulty in keeping the pavements dry; but if it is very flat, like Chicago, drainage is an important problem. As has been intimated, drainage is effected by what may be called an artificial grade in the gutters. This is very readily secured if a combination concrete curb and gutter is built, in which case the gutters can be built first directly to the desired grade, and the pavement can then be built to follow it. But when a curb only is built, great care must be taken to conform to the grade desired, as the curb itself will give no indication of the proper place to locate a summit. The gutters should drain directly to a sewer inlet, and the inlets should not be more than 300 ft. apart—200 ft. would be better. If it is assumed that they are 200 ft. apart and the grade is level, the summit would be in the center, and a run of 100 ft. would be made each way. If the gutter is 4 in. below the top of the curb at the summit, and a drop of 6 in. in 100 ft. is desired for proper drainage, the inlet would be

10 in. below the top of the curb. This is as deep as desirable; if made deeper, people are likely to stumble over the curb, and the crown is made so stiff on the ordinary street that it is difficult to pull a heavily loaded wagon into the center of the street. It might be claimed that the drop of 6 in. in 100 ft. is more than necessary, but any practicing engineer who has had experience with city streets will agree that it is not too much. It must be remembered that in the ordinary residence street, which is generally not cleaned oftener than once in two or three weeks, but is flushed nearly every day, a large part of the street sweepings accumulates in the gutters. Added to this is the sidewalk dirt, which is also swept into the gutter. In park roads and in streets which are swept and flushed every day, the rate of fall may be reduced, and good results will be obtained with a fall of 3 in. in 100 ft. This will reduce the sewer inlets by one-half; but the street cleaning must be of the best.

Another point to be noted is the drainage of the intersections. Ordinarily, this is a source of great trouble, especially if the intersecting streets are not at right angles. In this case the result of the crown intersections is a large flat space. The greatest care must then be taken in staking the grade for the concrete or other foundation in order to make it conform as nearly as possible to the theoretical intersections of the curves. Wherever possible, it is the writer's custom to raise the center of the intersection an inch or two higher than the established grade, in order to facilitate the drainage. An ideal intersection would have three inlets at each corner, placing one back of each of the property lines and one at the curb-line intersection, and bringing the pavement cross-walks flush with the sidewalks at these corners. It is not very often that this arrangement can be followed, as it is somewhat more expensive than the ordinary way of having one inlet at the curb corner; but the result in keeping a dry intersection would soon pay for it.

There is one more general point which might be mentioned, as it applies more or less to all pavements, and that is the foundation under the wearing surface. In late years, the concrete foundation has come into use almost everywhere for streets having a heavy traffic. Its usual thickness is 6 in., which, as a general rule, is sufficient; for streets of very heavy travel 8 in. is better. In streets in which necessarily many openings have to be made concrete is undoubtedly the

best foundation, but many residence streets and even business streets having a heavy traffic are but seldom disturbed.

Under such conditions, the writer is convinced that a thoroughly rolled stone base is the equal if not the superior of any concrete base. It keeps its line and grade just as well as concrete, if not better. As the heavy roller, used in compacting the stone, works continuously back and forth on this foundation for many hours, it will have every opportunity to discover any weakness in the subgrade. It may be said that the rolling of the subgrade should disclose this in any case, but, in addition to the subgrade rolling, the broken stone gets another tremendous pounding, to make a close, tight foundation, for which the concrete depends on its cement. Of course, a perfect concrete is inferior to no foundation, but, in ordinary work it is much easier to approach a perfect macadam foundation than a perfect concrete one. The writer believes that any engineer who has had experience in making either hand-mixed or machine-mixed concrete will agree with this.

In support of this general theory (which the writer is aware is opposed by many engineers), attention is called to the down-town streets of Chicago. Practically all are on a rolled foundation such as has been described, and, although the surface is badly worn, and the problem is complicated further by the poor construction of the car tracks, these streets are serviceable; and these pavements have been in service for from 15 to 25 years. The old granite blocks are rounded and worn, and will be replaced by a more modern surface soon; innumerable openings have been made in the streets, and this is an unfavorable location for such a foundation; but it is hard to believe that any concrete anywhere can show a better record. As an example of modern rolled streets, which show up very well, Harrison St., from Western to Rockwell, might be mentioned. This is an asphalt pavement on 6 in. of macadam, and was laid in 1903. It is in the best of condition, after 6 years of heavy traffic. Desplaines St., from Madison to Van Buren, of granite blocks on 6 in. of macadam, laid nine years ago, is a model surface to-day, though the traffic is very heavy. Wentworth Ave., from 55th to 63d Sts., was laid with brick on 6 in. of macadam, in 1897. Since then a conduit has been constructed in the street, and in 1908 the car tracks were reconstructed after being in bad condition for years, and yet the pavement

is serviceable to-day and is good for 5 years more. These are only a few examples. The only modification which would seem to the writer to be beneficial would be to put in an 8-in., instead of a 6-in., foundation on such streets.

In many places in which concrete has been used it would have been much cheaper to use a stone or gravel macadam. The writer knows of one small town in which the engineer provided a concrete base for a brick pavement, in spite of the fact that the street had been graveled for twenty years or more, and there was an abundance of good gravel in the vicinity.

In many cases a concrete foundation is a necessity, as, for example, in streets of very heavy traffic, streets having a bad foundation, such as wet clay, etc., and streets in which, undoubtedly, many openings will be made from time to time. As the concrete is generally from 6 to 8 in. thick, and is spread over a large surface, and not deposited in bulk, as in walls, abutments, etc., it is necessary to take more than ordinary care to obtain good concrete. A small sand pocket may make little difference in a large mass of concrete, but in a sheet like a pavement it may be the cause of a large hole which will ruin the wearing surface and extend with great rapidity under traffic. It is a vital necessity, therefore, to have inspectors at the delivery point and at the mixer, if one is used. If the concrete is mixed by hand, every care should be taken to see that it is thoroughly done, that too much water is not used in the mortar, causing the cement to be washed away, or too little, making a porous concrete.

The machine-mixing of concrete is one of the serious problems in street work. With many types of mixers, it is necessary, for a large part of the time, to have the mixer some distance from the work, such as at the next street intersection, etc. This means that each batch as mixed must be transported to the work in carts, wheel-barrows, etc. This is where the trouble comes in; the constituents of the concrete separate; in bottom-dumping carts, much of the "slush" or mortar, leaks out on the way; the heavier stone is shaken toward the bottom; and the net result is that the concrete is exceedingly uneven in quality when delivered. This trouble is largely eliminated by using a mixer placed close to the work and delivering the concrete on a board from which it can be shoveled into place. There are many places, however, in which it is very difficult, and sometimes impossible, to use such a

machine, and recourse must be had to hand-mixing or the distant machine. The writer would always prefer hand-mixing.

The proportions of the concrete are 1 : 3 : 6, using Portland cement, sand or limestone screenings, and broken stone, gravel, or crushed slag. The sand is standard torpedo. If limestone screenings are used, instead of sand, they must be clean, sharp, and free from dust or large flat pieces. There is no apparent difference in concretes made with sand or screenings.

One of the troubles in concrete foundations, caused by poor cement, poor mixing, etc., is the transverse cracks which appear later. These are especially bad for an asphaltic top, and cracks in this wearing surface are often caused by this fault and not by lack of bitumen, but longitudinal joints or cracks in the concrete appear to affect the asphaltic surface but little.

During the past two years, practically all the car tracks in Chicago have been reconstructed. This involved slight changes in grade in many cases, and often, on an asphalt street, the pavement was completely cut out half way back to the gutter. In laying the new concrete no particular effort was made to obtain a good bond with the old, other than by brooming a little and spraying it with water. It is probable that in most cases there is no real bond between the old and new concrete. After the concrete had set (generally in 4 or 5 days) the new top was laid. Some of this construction has now been down for 2 years, and no signs of cracking or failure have been seen.

The Wearing Surface.

In this paper no attempt is made to discuss macadam pavements of various types, as this subject has been treated very thoroughly in a recent paper by A. H. Blanchard, M. Am. Soc. C. E.* Strictly speaking, macadam is not suitable for a city pavement, but only for parks and light residence streets.

Brick Paving.—Ordinary vitrified brick is one of the most popular paving materials in small cities, but is not as popular in the larger ones. It requires a good foundation—as does any pavement—and the only problems in connection with it are the quality of the brick and the kind of filler. All block pavements require a sand cushion or bed of 1 or 2 in. under the blocks.

* *Transactions*, Am. Soc. C. E., Vol. LXI, p. 445.

In regard to the foundation, various substitutes, cheaper than a macadam or concrete base, have been used, but not with any great success. In some small towns having a sandy subgrade the bricks are laid directly on the sand after rolling the subgrade, but the pavement soon wears into ruts under any large amount of traffic. A base course of bricks instead of concrete has also been used as a foundation, but, if laid properly, this is as expensive as concrete, or more so, and is not as good.

The quality of the brick is determined by water absorption, specific gravity, and abrasion tests. To these might properly be added a test of the uniformity of shape. It happens frequently that bricks which are otherwise good have been warped so much by the firing that they have to be rejected on account of their shape. If a uniformity test were applied, and the provision were made that not more than a certain percentage of the bricks should depart from the required shape, the pavement would undoubtedly be improved. The form of this provision might be as follows:

"Not more than 5% of the bricks shall depart a greater distance than $\frac{1}{4}$ in. from the standard block furnished by the manufacturer. A greater percentage than this shall be cause for the rejection of the entire lot from which the samples tested were taken."

It is not possible to give exact sizes for paving brick, as those of different manufacturers vary slightly, but it can be specified that they shall not be smaller than a certain size, say 3 by 4 by 8 in., or larger than $3\frac{1}{2}$ by 4 by 10 in., and this is generally done.

The water absorption test is very important, as it measures the porosity of the brick. If the weight of water absorbed is more than a very small percentage of the weight of the brick, the latter is almost sure to go to pieces in two or three years under the freezing and thawing action of winter. The Chicago specifications limit the quantity of water absorbed in 72 hours to 3% of the weight of the brick, and some of the engineers desire to reduce this to 2 per cent. This would probably be done if it were not for the fact that so many bricks are rejected under the 3% clause; for it is thought that at the 2% rate no brick at all could be obtained.

The specific gravity required is generally about 2.1. The various manufacturers have no trouble in meeting this specification, and it has been found by experience that this gives a good heavy brick.

The abrasion test is also very important. A brick may be dense and heavy, but so brittle that it will not stand pounding. For this test the rattler is used. It consists of a steel cylinder, about 28 in. in diameter and 20 in. long. The sides are composed of steel staves having open spaces between to permit the escape of broken fragments. The size of the charge is about 15% of the volume of the rattler. Inside the rattler a number of pieces of loose heavy castings are placed. The rattler makes 30 rev. per min., and is run for an hour; bricks which lose more than 20% of their weight are rejected.

Disregarding the quality of the brick, the point over which there is most controversy is probably the filler. Three fillers are in general use: sand, bitumen, and cement grout. Sand is the oldest and cheapest form, but is not now used to a great extent. The objections to it are that it does not make the joints water-tight, and that flushing the street forces the sand out of the joints and loosens the brick.

The most popular material is either a cement grout or a bituminous filler such as coal-tar, pitch, or asphalt. The writer believes that, as far as wear is concerned, there is little to choose between tar, asphalt, and cement grout fillers. In favor of the cement grout, it is urged that it makes the pavement approach a monolithic whole, protects the edges of the bricks, and is most easily cleaned and kept clean. Its chief objections are that it increases the noisiness of the pavement, and that after grouting, the pavement must be undisturbed for several days in order to set thoroughly. In any street of large traffic, it is nearly impossible to comply with this last requirement, and a corps of police would be needed to keep the traffic off the apparently finished street. For this reason alone, the authorities in Chicago have entirely abandoned grout, and now use a bituminous filler exclusively.

The grout is generally made in the proportions of 1 part of cement to 1 or 2 parts of sand. It is generally maintained by engineers that it must be carefully mixed in a suitable box, and that every precaution must be taken to prevent the separation of the sand and cement. From his experience, the writer believes that this idea is good, but its practical attainment is difficult and unsatisfactory in any form of box. Formerly, it was his custom to insist on the box, but there was so much trouble that one of the contractors was finally permitted to try his own scheme. He made a thorough mixture of the dry sand and cement and swept it into the joints, thus filling them about half

way to the top. A spray of water was then applied and the grout was formed in the joints. Plenty of water was used, care being taken that it was not forced directly into the joints so as to wash out the mixture.

For a second course, sand and cement were mixed dry in piles directly on the pavement, and then the hose was turned on it. By using squeegees and the hose, this was thoroughly mixed, and the whole was then spread over the pavement and into the joints. The results have been of the best. During the past two years, there has been occasion to examine several openings made in pavements grouted in this way, and there is yet to be found a bad joint. In one particular case, where the adjoining street was afterward paved with asphalt, and the brick pavement at its intersection was exposed and, for about half the width of the brick, undermined, the grout joint held it up in spite of the fact that the material wagons were constantly driving over this intersection.

The advantages of a bituminous filler are that it tends to make the pavement less noisy, and also makes any repair easier than when grout is used, for it is very difficult to make openings in the latter if it has been properly applied.

The bituminous filler is either a pitch, which is the result of the distillation of coal-tar; or some asphaltic compound. The name pitch simply means coal-tar from which the water and lighter oils have been driven off by distillation, the product being hard at ordinary temperatures. If the distillation is carried too far, parts of the heavy oils are also driven off, and the result is a hard, dense product, which cracks very easily, and is not suitable for the purpose.

The pitch filler is either poured on the pavement from buckets and squeegeed into the joints, or else poured directly into the joints with a special bucket. The latter method is a little more expensive, but is the better, as the pitch on top of the brick is a nuisance.

The asphaltic filler is probably better than the pitch filler, as it does not crack so much in cold weather, nor does it lose its oils as rapidly as the tar products. In other words, a filler having an asphaltic base is more stable than one having a coal-tar base; but, as a general rule, it is also about twice as expensive. It is always applied with a special pouring bucket, and is never squeegeed.

If a grout filler is used, some provision should be made for expansion. Generally speaking, an expansion joint of $\frac{1}{2}$ in. along the gutter,

and ½ in. longitudinally every 50 ft., will be ample. If there are street-railway tracks, it is advisable to lay the bricks for about three courses longitudinally with the street, and use a bituminous filler, not on account of expansion, but because of the incessant vibration caused by the cars, which tends to break the bond of the grout.

Secretary Blair, of the National Paving Brick Manufacturers Association, recommends the total elimination of the transverse expansion joint in brick paving, maintaining that the longitudinal joint is sufficient, but the writer is not prepared to agree with his reasoning.

Each of the various companies promoting tar, asphalt, and cement grout fillers insists that its product is the best. During the past three years the writer has made openings in many brick pavements in which a pitch filler had been used, and has nearly always found the joints well filled, even when the pavement was many years old. The same could no doubt be said of the asphaltic filler, but it costs more than twice as much as the pitch, and it has not been proved that it will really give any better service, though from its nature it should be somewhat superior to pitch.

Although cement grout at first makes a smoother surface, in the course of three or four years the grout in the joints is broken by the blows of countless horse-shoes, and after that there is practically no difference between its appearance and that of a tar-filled street. The joints then rapidly wear down, and become wide and rough.

An example of exceedingly good service given by brick may be seen on La Salle St., from Randolph to Madison Sts. This pavement has been in service for 14 years, and it is still usable, though nearly worn out. The traffic on this street is enormous, and innumerable openings have been made.

The principal objections to a brick pavement are its noisiness on residence streets and the wear of the edges on heavy traffic streets. Its advantages are that it is easily cleaned, is not affected by the weather, and, under proper conditions, has a very long life.

Stone Block.—The next grade of pavement, considered purely from a wearing standpoint, is the stone block. It is made with dressed blocks, generally about 5 in. wide, 5 in. deep and from 8 to 10 in. long. The block should be a hard stone of medium grain, which will resist the weather and not wear smooth and round. Sandstone of various kinds, granite, and what might be called "near-granite," such

as quartzite, which is really a sandstone, rhyolite, etc., are used. Various kinds of artificial blocks, such as iron slag, etc., have also been tried, but, in this part of the world, at least, have not met with much favor.

Much better stone block pavements are now laid than formerly. This is due largely, if not entirely, to more careful dressing, which makes a smoother and more regular block; and more care is taken in laying them. Formerly in Chicago the blocks were only roughly dressed, and were laid with wide joints. The result was that the edges were very much rounded under traffic. No investigation was made as to whether or not the blocks became slippery under traffic. Blocks which in the past have worn badly will not now be accepted. Samples of the so-called granite block to be used are required by the authorities when a bid is made, and the location of the quarry from which the stone is taken must be stated.

Although called granite blocks, it is not insisted that they shall be a true granite; but they must be what is commercially known as a granite, and must show a record of efficient service. The result has been, that the stone block pavements laid during the past 5 or 6 years, are very good, being nearly as smooth as brick, and of course much more durable. Examples of granite paving which the writer considers very good, may be seen on La Salle St., north of Lake St.; on North Ave., west of Clybourne Ave.; on Michigan St., east of State St.; and on many other streets. Practically no granite has been laid in "The Loop" district for many years, as it has come to be recognized that it is too noisy for an office, retail, and shopping district, and property owners are willing to pay for the renewal of a quieter pavement.

A sandstone block in the proper location has many points in its favor, but, like everything else, it shows up badly when in the wrong place. It is not suitable for streets of exceedingly heavy traffic, such as "The Loop" district. It does not get as smooth or rounded as the harder granite, but it wears rapidly, and in holes. If it wore uniformly it would be ideal, as far as service is concerned, but under Chicago's enormous traffic it fails. An example of its use may be seen on Washington St., just west of La Salle St. Kettle River sandstone was used, and it has been in service about 4 years. It has been necessary to patch it with granite. If this had been laid on a

secondary business street, such as the principal street of an outlying district, or on a street where the business is principally wholesale, it would probably have given good satisfaction for many years. Even under its present conditions, it has not worn smooth, and the joints are still close and regular.

Considering quartzite as a sandstone, though it is generally called granite, the record is much better. It has a high crushing test, nearly equal to granite; is somewhat coarser; does not wear as smooth, and cuts very nicely. Much has been laid in Chicago, with good results. Most of it came from Sioux Falls, S. Dak. Stone from about a dozen quarries has been used in Chicago, but nearly half of them have been eliminated during the last 4 years, either because the granite was too soft, or was so hard that it could not be properly dressed.

It is generally recognized that a good granite pavement will outwear all others, but it is very noisy indeed, and is being relegated to wholesale and factory districts, where the traffic is of the heaviest truck character, and noise is a secondary consideration. It is also rather unsanitary, and is expensive to keep clean.

The filler is usually of mixed gravel and tar. Grout has been tried with success, as far as the pavement itself is concerned, but has been abandoned on account of traffic conditions, it being practically impossible to keep the roadway blocked for a week in order to let the grout dry.

Wood Block.—One of the most popular wearing surfaces of the last few years is obtained with preserved wood blocks, generally called the creosoted wood block pavement. Its advantages are that it is smooth, noiseless, sanitary, and very easily repaired. About its only disadvantage is that it is slippery. It has been used for many years abroad, and has given good satisfaction. In America it has to overcome the bad name given to wood block pavements by the old untreated cedar block pavements, generally laid on plank alone without any other foundation, with results which can easily be imagined. The modern wood pavement is entirely different, the blocks being rectangular, instead of cylindrical, and always laid on a good foundation.

The cycle of operations by which the block is prepared is as follows: The seasoned or partly seasoned timber is cut into 4 by 4-in. blocks from 5 to 10 in. long. These are placed in a vacuum chamber and the air is exhausted. The wood preserving oil is then admitted

to the chamber, and forced by pressure into the wood. It is desirable that the blocks shall be evenly impregnated with the preservative all through, especially if they are to be laid on a street of moderate traffic, as they will probably rot out before wearing out.

The Chicago specifications first required that 12 lb. of oil be forced into each cubic foot of timber; later, this was changed to 16 lb., as it was found that 12 lb. did not fill the fibers. Experience has shown, however, that southern long-leaf yellow pine requires from 18 to 21 lb. of oil to impregnate the fiber thoroughly, and the question then arises whether or not the wood is injured by the pressure necessary to inject the oil. It takes 6 hours for a charge, and a pressure of about 170 lb. is required to force the oil into the wood. About the same pressure is required for a 16 or 20-lb. treatment, the latter simply taking a longer time. If a lighter pressure were used, say 100 lb., the time required for the oil to impregnate long-leaf yellow pine thoroughly would probably be from 16 to 20 hours. It might be noted here that Michigan Ave., south of Van Buren St., was laid with blocks having a 10-lb. treatment some 10 years ago, and is in perfect condition to-day. This street has a heavy boulevard traffic. Only a very few openings have been made.

It has been generally specified that southern long-leaf yellow pine alone shall be used. It is quite possible, however, that this is a mistake, and that many other varieties of pine, such as tamarack, Norway pine, etc., which are cheaper, could be used, and with excellent results. It is probably a little more difficult to season them properly, but it is claimed by the tamarack promoters that it can be just as well done, that there are fewer large knots, and that there is less likelihood of checking around the circumferences of the rings of the timber. In the new specifications, several kinds of timber, such as tamarack, Norway pine, black gum, etc., are specified.

One of the troubles with blocks having a cross-section of 4 by 4 in. is the fact that, owing to the accumulated oil on their surfaces, it is sometimes difficult to tell which is the proper way to lay them. It is obvious that they should be laid on edge, for, when laid with the grain or flat, they very soon wear. In Chicago streets, therefore, in spite of rigid inspection, some of the blocks have been placed on their sides. The way to remedy this is to use a block having a different width and depth, such as $3\frac{3}{4}$ by 4 in.

It has been stated that in New York City blocks 3½ in. deep are used. In Chicago their depth is 4 in. In contrast with these figures, the practice abroad is to use blocks having a depth of from 5 to 7 in. New York City has been criticized severely by various engineers for using such shallow blocks. In a recent report* made to the City of New York, upon paving abroad, it is stated that in Paris the blocks, when badly worn, are taken up, the worn part is cut off, and the blocks are then relaid. In spite of this, their average life is only 9 years. Chicago pavements do better, for there are good creosoted pavements which are 10 years old, and, as far as can be seen, are good for 10 years more. The probable reason for the short life in Paris is stated to be the excessive amount of sprinkling.

It is also probable that another reason is the superficial treatment the wood receives. According to American ideas, the blocks are hardly treated at all, simply being given a hot bath in the preserving oil. It would seem that French engineers now recognize that this was a mistake, for, in a paper entitled "The Future of Exotic Wood as Applied to Pavements," presented to the First International Road Congress by M. Louis Mazerolle, Engineer of Bridges and Roads, of Paris, and translated for the City of New York, it is noted with approval that wood blocks, creosoted under pressure, are still in good condition after a service of 15 years.

After reading of various French methods, the American municipal engineer will certainly come to the conclusion that American construction is decidedly better, but that the French maintenance is very much superior.

Another point is the fact that the larger the blocks the more difficult it is to impregnate them completely. Therefore, when they are sawed off, a large part of the resulting surface will certainly be of nearly untreated wood. Even if the block is turned over, it will probably begin to rot in a very short time. This course of treatment, therefore, will probably prolong the life of the pavement only 2 or 3 years, and then the process will have to be repeated. Taking into account the increased cost of the blocks, the necessarily heavier treatment they must receive to impregnate them, the fact that high pressure may injure the fibers of the timber, while if a low pressure is used, the time required to impregnate the timber will be prohibitively in-

* *Engineering News*, Jan. 21st, 1907.

creased, the cost of repairing the deep blocks by sawing and the likelihood of inadequate results, it is quite probable that American practice is better. It might be mentioned that, in the same report, New York was also criticized, by implication, for the lack of depth of its granite blocks. This is not good reasoning. It is not probable that a granite pavement ever failed because of the shallowness of its blocks. Granite wears out because of the rounded edges, becoming slippery, etc., thus making a bad footing. A deep stone will certainly not make a better pavement; what is required is a better dressed block and more care in laying.

Returning to wood blocks: It has been the custom in Chicago to lay them very tight, in order that the surface may be sealed as much as possible. There has never been any trouble until the past year, when the tightness was carried to an extreme, sledges being used to make the joints as close as possible. During the winter of 1908-09 the pavement heaved. In spite of this, it is probable that no trouble would have occurred had the timber been of the proper kind. The heaving was undoubtedly caused by water absorption and then freezing and thawing. In one case tar was used as a filler, and in the other case sand.

Experiments made by the Kettle River Quarries Company* have demonstrated that, with the treatment specified (16 lb.), there is absolutely no water absorption, even when the block is left for weeks in water. There is not much expansion in wood. Trautwine states that white pine expands 1 part in 440 000 for each degree Fahrenheit. As expansion joints are always provided when such a filler as sand or grout is used, and as none is needed with a tar filler, and considering the fact that, in each case where failure occurred, the wood was supplied by a company other than that which had previously furnished it, the conclusion is inevitable that the fault was in the treatment or the timber. Though the work was not done under the writer's supervision, he is of the opinion that the fault was in the timber, in that it was of too rapid a growth, having large cells, etc., which could not be well sealed. Undoubtedly, also, in one case at least, the blocks were laid unnecessarily tight, for while they should be tightened as much as possible with bars, they should not be pounded to refusal. This point is taken up later.

* Discussion by Walter Buehler, Assoc. M. Am. Soc. C. E., at the meeting of the Illinois Society of Engineers and Surveyors, February, 1909.

In Chicago, the Kreodone-Creosote oil, produced by the Republic Creosoting Company, has always been the standard, and the results have been satisfactory. In a paper presented before the Illinois Society of Engineers in 1908, Mr. John B. Hittell, Chief Engineer of Streets, Chicago, shows that a large percentage (about 9%) of the oil of a half block volatilized in the first three days it was exposed in the even temperature of a closed room. That more than this is lost on the street is almost certain, and the action is very evident from the appearance of the blocks piled on the street and waiting to be used. In the strong sunlight they very soon begin to check badly in all directions, and the action continues until they are laid in the pavement, filled, and covered with sand. After they are in service, there is little apparent loss.

It was early recognized that to use as a standard the treatment advocated by any one company was anything but scientific, but at that time the city had no adequate laboratories in which to try out and analyze the different processes. Since the installation of an asphalt laboratory in 1907, however, it is in better shape, and the specifications have now a more scientific basis. The principal points are as follows:

The quantity of oil used is 20 lb. per cu. ft., but it is the writer's belief that, for the reasons previously stated, this is unnecessarily large. The oil must be a pure, coal-tar product, free from any adulteration; it must not contain any petroleum, nor more than 5% of matter insoluble in benzine. Oil obtained or partly obtained from water-gas tar, or oil tar, will not be accepted.

The specific gravity of the oil shall be at least 1.1 at 25° cent.

In the matter of volatilization of the oil, which, as has been stated before, was found to be a very serious one, the following provisions have been made:

Up to	150° cent.	Nothing must come off.
" "	210° "	Must not volatilize more than 5 per cent.
" "	235° "	" " " " " 15 " "
" "	315° "	" " " " " 40 " "

It is the writer's opinion that it will be exceedingly difficult, if not impossible, to enforce rigidly these percentages, but, undoubtedly, an attempt should be made to approach this standard.

Another very important point, which has been mentioned before, is the large size of the cells and hence of the annular rings of the timber; this is also noted in the specifications, and the provision is inserted that, in long-leaf yellow pine, the number of annual rings shall be not less than seven per inch, measured radially from the center. This provision will bar out first-growth timber which has had too rapid a growth, and in which the cells are so large that it is difficult to seal them. As previously mentioned, one of the apparent failures was probably due to the freezing and thawing in the cells of the timber.

One of the troubles with creosoted block pavements has been caused by the brow on streets having car tracks. Such streets, ordinarily, have more than the average amount of traffic, as they are generally the business streets. The Chicago specifications provide that the blocks shall be placed on a sand cushion of 1 in., but even if the greatest care is taken with the concrete, it is not possible to get an absolutely uniform surface, and the sand bed will vary from $\frac{1}{2}$ to $1\frac{1}{2}$ in. As the blocks must be rolled after they are laid, the sand bed must be graded so that when first laid the blocks will project somewhat above the car tracks. When the street is rolled the blocks are forced down, and afterward the traffic pounds them down still farther. In the course of time, if the blocks have not been set high enough, or the sand cushion is too deep, they are pounded down until they are considerably below the edge of the rail, making an ugly bump in the street, and a depression in which water will stand along the edge of the rail. Fig. 2 shows this condition; it should be noted that the shoulder, *A*, is too high, and *B* is too low. Water collects at *C* and disintegrates the pavement.

On the other hand, if the blocks are set so high that they never pound down to the top of the rail, the pavement along the brow will fail in a few years from brooming along the edge, thus tending to tip at the rear end of the block. Fig. 3 shows this condition. Both *A* and *B* are too high, the block is ground and distorted at *C*, and the accumulation of water at *C* tends to destroy the pavement.

In Figs. 2 and 3, *C* is the controlling point, and must be higher than any point beyond it.

Either of these cases can be avoided only by the greatest care in shaping the concrete foundation at the brow and shoulder. If the blocks are 4 in. deep, and 1 in. of sand bed is used, the concrete should be $4\frac{3}{4}$ in. below the top of the rail. Sand $1\frac{1}{4}$ in. deep will compact to 1 in.

The block will then be $\frac{1}{2}$ in. above the rail when first laid. Rolling and tamping will reduce this to $\frac{1}{4}$ in. and the traffic, after a shorter or longer time, will pound the brow down to the rail level.

It seems to be a custom in Chicago and elsewhere to require that the courses be laid at an angle with the line of the roadway, generally, 45 degrees. The result is an enormous amount of waste, because every course must be ended and finished by a skew block, and the regular blocks must be cut at an angle to make this skew. These skew blocks, on account of their sharp, triangular ends, are also particularly susceptible to wear, and must be and are a point of weakness. It seems to the writer to be the height of absurdity to continue this 45° or similar angle. It is simply following the custom of brick paving, without any of the same reasons. In brick paving the joints are comparatively wide, therefore it probably saves a good deal of wear on the edges to lay the bricks at such an angle, but a preserved wood pavement is as smooth as asphalt, has practically no open joints, and will certainly be cheaper and better if the courses are laid at right angles to the line of the street.

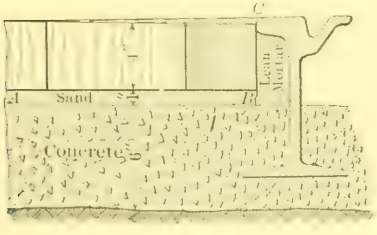


Fig. 2.

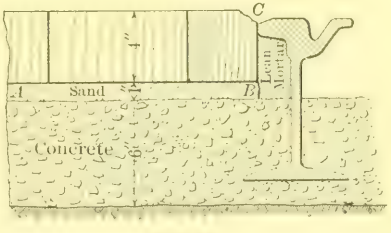


Fig. 3.

Asphalt.—Asphalt is undoubtedly the most popular pavement in the larger cities of the country to-day. Approximately 50% of all the pavements now being laid in Chicago are asphalt. The reasons, of course, are its noiselessness, cleanliness, ease of repair, good looks, and comparative cheapness. It costs less than brick, only 60% as much as creosoted block, and about 50% more than standard macadam. It is, however, probably the most technically difficult pavement to lay, and should never be laid without expert advice.

As a material, it has been abused in past years more than any other. It has been laid by irresponsible firms, without adequate advice, has been laid on streets for which it was not adapted, and the result

has been many failures. The authorities, however, have come to the understanding that it needs the best of supervision and inspection, and, under such conditions it has become nearly the ideal paving material. Formerly it was the custom to accept the guaranty as being all that was necessary to get good results. The city engineer had very little knowledge of the material, beyond that imparted by the companies doing the work. It is different now; Chicago, like other cities, has established an asphalt laboratory, and samples from each job are examined daily by a competent chemist.

The modern asphalt pavement is composed of the foundation; the binder course, consisting of small crushed stone, of 1 in. in diameter and less, mixed with asphaltic cement, intended to give body to the top; and the top or wearing surface, consisting of asphaltic cement and a filler of fine sand and Portland cement.

The specifications were formerly very general indeed, no percentage of bitumen was specified, nor was any attempt made to regulate the manner of preparing the refined asphalt, the flux (or oil to be mixed with the refined asphalt, to render it a workable cement), or the sand grading. Now all this is looked into closely. It is specified that crude natural asphalt shall not be heated to a temperature of more than 400° Fahr. in order to drive off the water. It is also specified that at least 98½% of the bitumen which is soluble in cold carbon bisulphide shall be soluble in cold carbon tetrachloride.

Mr. L. Kirshbaum, Asphalt Chemist for Chicago, shows by detailed experiments* just why this provision was inserted, as it was objected to by some of the native asphalt companies. Suffice it to say that he demonstrates that the carbenes (which is a term applied to those asphalt bitumens which are more soluble in carbon bisulphide than in carbon tetrachloride) are an evidence of lack of care in the preparation of asphalt from the native asphaltic oils, and for that reason the clause is justified.

Formerly the use of Texas or California asphalts for the wearing surface was not permitted, but of late years it has been demonstrated that if these heavy oils, having an asphaltic base are prepared properly, they are the equal of any of the Trinidad or Bermudez asphalts. Therefore the Chicago specifications now provide for the use of Cali-

* In an article in *The Municipal Engineer*, December, 1908.

fornia asphaltic petroleum, distilled at a temperature of not more than 600° Fahr., giving a residue not harder than 40° penetration at 77° Fahr., by the Dow machine. The requirement as to carbon tetrachloride, of course, applies to this asphalt in common with the others, together with a test to show that most of the light oils have been driven off in distillation. The requirements are that the sample tested shall not lose more than 5% by weight, nor shall the penetration be reduced more than 50% when heated for 7 hours at a temperature of 325° Fahr.

The fluxing material is also carefully specified, and as carefully examined for penetration, and for water, coke, and other impurities. The whole idea is to prepare an asphaltic cement which will not become too soft in summer or too brittle in winter. Equal care is taken with the sand grading; the sizes of the grains must be such that practically none shall pass a 200-mesh sieve, all shall pass a 10-mesh sieve, the bulk of it shall pass a 40-mesh sieve, while a smaller part shall pass an 80-mesh sieve.

The binder, also, is now what is called a tight material, as opposed to the old open binder. In the tight mixtures the voids are nearly filled with small stone or torpedo sand, in the open binder this stone was of nearly uniform size, about ½ in. The result of this was many so-called "fat" spots, or a concentration of the asphaltic cement, which had a tendency to rise through the wearing surface and make a weak place in the pavement, and this invariably wore into a hole. The tight binder mixture prevents this, as the asphaltic cement is distributed more uniformly through it. It also has an advantage which is immediately apparent. It may be teamed over, without breaking up under the traffic. This is necessary because the material for the top or wearing surface must be delivered in wagons over it. The old open binder very often broke up under such traffic, and then became no better than the old cushion coat used 20 years ago. It is too soon as yet to see the improvement in the wear of the pavement, as thus inspected and prepared, but the writer thinks he is safe in saying that not a single pavement laid during the past 2 years, when this care has been taken, has cracked or shown signs of failure.

Asphalt, according to experience in Chicago, is not suitable for streets having the heaviest commercial traffic, such as those in "The Loop" district, as the best pavements laid thus far have shown a

tendency to wave and wear rapidly. As an example of this, six blocks on State St., from Randolph to Jackson Blvd., may be cited. This stretch was laid by one of the most responsible companies in the business. It employs a chemist constantly to look after the mixtures. Part of the pavement was under a 10-year guaranty, and part under a 5-year guaranty. On both stretches the guaranty will expire within a year. This street has an enormously heavy traffic, being in the heart of the city's retail shopping district. A repair gang is kept busy patching almost constantly and the surface is quite wavy. On account of the character of the traffic, every effort was made to get a perfect construction, but while the pavement is kept in fairly good condition with constant repair, the surface cannot be called a success on such a street.

Again, property owners, in their enthusiasm for a nice, smooth pavement, have called for asphalt pavements on streets where the traffic is so light as to be almost nothing. Here, again, the result has been bad. Lack of traffic, apparently, is nearly as fatal as too much. The surface cracks, shrinks, and dries up, and in three or four years is a mass of unsightly cracks. Between these extremes the results have been best. Many examples of this may be shown: La Salle Ave., from Chicago Ave. to Clark St., which is 15 years old, has a fairly heavy traffic, and, although it is rather badly worn now, 80% of its surface is in good condition. Dearborn Ave., between the same limits, is 13 years old, and is in excellent condition. Indeed, one block of this is more than 25 years old, and can hardly be distinguished from the remainder. Both these streets have been repaired within the last year.

As a general rule, however, the impossibility of making repairs, except with a special plant, makes asphalt a not altogether desirable pavement for a small town, unless that town is very near a large one. The amount of work to be done in such a town is not enough to pay the contractor to keep a permanent plant there, and therefore he will make repairs but once a year, if possible, and bad places in the pavement may go several months before being repaired.

In Chicago nearly all the various brands of asphalt, both native and imported, such as Trinidad, Bermudez, California, Texas, etc., have been used, and under proper supervision they are all apparently good. For some of them too short a time has expired to be sure of results, but none has shown signs of failure as yet.

While streets may be seen in Chicago where asphalt has been in service for 20 years or more, it is probable that, after about 10 years, there is practically no "life" left in the wearing surface. Practically all the oils, both light and heavy, have been volatilized by that time, and a sample of the top may be disintegrated between the fingers, producing nothing but clean sand, and leaving absolutely no stain on the fingers. The blackness may be seen in small grains, and is probably the "coke" of the old asphaltic cement. The writer has made this experiment many times with the same result. On streets with light traffic, the pavement will probably be good for some years after that, but on streets with heavy traffic the surface goes to pieces rapidly when this stage is reached, and even constant patching will not keep it in good condition. An example of this may be seen in North Clark St., between Kinzie and Division Sts. This was paved in 1897, has a very heavy traffic, and has been repaired every year, for the past few years. In the fall of 1908, it was thoroughly repaired, but at present (April, 1909) it is again in bad shape, new holes constantly appearing.

Bitulithic pavement, nearly allied to asphalt, has become popular of late years, and has much to recommend it. It is patented, and for that reason cannot be laid in Illinois by special assessment. In Chicago, however, it can be and is laid by the park boards, which do not work under the same law. It has been very successful. It consists of asphaltic bitumen mixed with a carefully-graded aggregate of crushed stone. The mixing and grading of the stone are done by special patented machinery, and, when the mixture is delivered on the street, it is spread and rolled like ordinary asphalt. On the surface is spread a flush coat of bitumen, and chips are then spread over this flush coat and rolled in. The result is a fine, even surface, not as smooth as asphalt, but in other respects very similar. On account of the fact that the city cannot legally lay such a patented pavement, none has been laid under the writer's supervision, but from observations of pavements put down by the park boards, his opinion of it is very high. One stretch of ¾ mile, on Sheridan Road from Irving Park Blvd. to Byron St., which he has had under observation, has been down 4 years, and has been very satisfactory. The street has an exceedingly heavy boulevard traffic, but there have been practically no repairs, the surface is in excellent condition, and has not cracked, or become slippery. The cost of the bitulithic pavement is slightly greater than that of

asphalt, and, if equal precautions are taken in the selection of the bitumen, there is no reason why its life should not be as long, if not longer.

Another adaptation of this same principle—which may or may not be an infringement of bitulithic patent—is the so-called "Mineral Rubber" pavement, now being laid by the park boards in Chicago. It consists practically of run-of-crusher stone, 1½ in. and less, mixed with asphalt. No particular attempt is made to grade the stone, and the ordinary brands of asphalt may be used. A flush coat of asphalt is spread, and the top is sprinkled with torpedo sand. In Chicago it has been mixed with portable mixing machines and laid on the street in sections across the roadway. These sections are about 8 ft. wide, and are separated by wooden strips. The surface has been very good, smooth and not slippery, but is somewhat uneven, due probably to lack of rolling. The writer is opposed to the strip method, and more in favor of continuous construction. Joints are always points of weakness, and he sees no good reason for making an exception in this case.

Miscellaneous Pavements.—Various kinds of concrete pavements have been laid with some success in Chicago. These consist essentially of a bottom course of about 5 in. of ordinary 1:3:6 concrete, and a second course of richer concrete, laid before the first has set. The whole is then finished with a flush coat of rich mortar, which is smoothed off and grooved into rectangles of about 5 by 10 in. The greatest care must be taken to get proper expansion. A system of streets in the vicinity of the Sears, Roebuck Company's plant on the West Side of Chicago, only 3 years old, is very badly cracked, both longitudinally and transversely, on account of expansion. In places this action has nearly overturned the curb. Wide cracks extend down the center of every street, and the pavement is in bad shape. Plate XLVI is a photograph of one of these streets. No comment is necessary. A similar pavement, laid on Ohio St., near the Lake Shore, which is only 1 year old, but in which adequate expansion joints were provided, shows no signs of failure.

These pavements are very desirable in many ways, being easily cleaned and easily repaired. They have been used in several alleys. The surface has not worn too well, in some places, under heavy traffic. One alley in the rear of the Lexington Hotel and another at 31st St.

CRACKS IN CONCRETE PAVEMENT IN A CHICAGO STREET.

and Wabash Ave., have shown decided signs of failure, the joints being battered down and the surface disintegrating, although they have been finished for less than 1 year. They were laid in two courses, a botttom course, probably of 1:3:6 concrete, and a top course of rich mortar, probably 1:2. These are extreme examples, and the pavement should not be condemned on these alone. They were not laid under strict supervision, being done by what is called private contract.

Various types of patented pavements, of wood, concrete, etc., have been tried from time to time, but the trial pavements are too limited in area to enable one to comment intelligently on them.

TABLE 1.—Miles of Pavement of Various Kinds Laid in Chicago by Special Assessment, from 1903 to 1908, and Average Prices per Square Yard.

Year.	Asphalt.		Brick.		Creosoted Block.		Granite.		Macadam.	
	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.
1903..	49	$2.46	7	$2.65	0.03	$3.25	3.5	$4.33	20	$1.38
1904..	43	2.50	5.5	2.91	0.57	3.52	4.6	4.06	31	1.38
1905..	48.7	1.985	6.6	2.26* 2.62‡	0.20	2.65†	4.0	3.60	38.9	1.22
1906..	52.4	1.88	5.1	2.38* 2.64‡	0.75	3.23	3.9	3.59	43.5	1.15
1907..	57	2.17	3.25	2.26* 2.62‡	0.90	3.78	5.5	3.93	18.9	1.21
1908..	38	2.09	10.6	2.35* 2.51‡	4.7	3.93	6.5	4.06	16.7	1.25

* All streets.
† One job, blocks laid on old macadam.
‡ All alleys.

Table 1 shows the various classes of pavements laid each year by special assessment, and the average prices per square yard. Asphalt is by far the most popular in the city districts. In the outlying districts, macadam is naturally the most popular, on account of its low first cost. The prices include such grading as is necessary to prepare the subgrade, where a street is being repaved, but do not include any large amount of grading in any case. The price per square yard includes the pavement complete, that is, the foundation and the wearing surface, but not the curbs, sewer adjustments, etc. All pavements are under a 5-year guaranty except macadam, which is under a 2-year

guaranty. It will be noticed that the price of brick is very high. This is because brick has been used almost exclusively as an alley pavement in recent years. All the foundations (except macadam) are of Portland cement concrete, 6 in. in depth. The macadam consists of an average of 12½ in. of material, 9 in. being limestone in two courses and 3½ in. being granite in one course.

TABLE 2.—QUANTITIES OF PAVEMENT IN CHICAGO AT THE PRESENT TIME, 1908.

	Miles.	Approximate Yardage.
Asphalt	416	6 955 000
Asphalt blocks	2	30 000
Rock asphalt	0.6	10 000
Brick	107	1 260 000
Cedar block	452.50	8 100 000*
Concrete	3	30 000
Creosoted block	7.25	130 000
Granite	63	1 260 000
Medina stone	1.15	20 000
Macadam	552	9 000 000
Novaculite macadam	2	32 000
Slag macadam	6	96 000
Total	1612.5	26 915 000

* Practically all the cedar, some 8 000 000 sq. yd., is now worn out and useless; probably 90% of the remainder is in fairly good condition, and 10% is in bad condition.

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PAPERS AND DISCUSSIONS

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THE PURIFICATION OF THE WATER SUPPLY OF STEELTON, PENNSYLVANIA.

By James H. Fuertes, M. Am. Soc. C. E.

To be Presented October 6th, 1909.

The plant for the purification of the Steelton water departs, in several particulars, from previously constructed types, and embraces features of some novelty. The interest expressed by engineers and municipal authorities who have visited it, and seen it in operation, inclines the writer to believe that, although the plant is not a large one, a description of the conditions leading to its design, as well as an account of its construction and the results thus far obtained in its operation, would bring out an instructive and useful discussion of certain subjects on which more information is desirable.

The plant is a slow sand filter having a capacity of 3 000 000 gal. of filtered water daily; and among its novel features are:

1. The use of coarse-grained filters instead of subsiding basins in the preparatory treatment of the water;
2. The small area required—about one-fourth of that ordinarily required for a slow sand filter plant of equal capacity;
3. The system of handling and caring for the sand removed from the slow filters during scraping;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

4. The system of determining, measuring, and applying the chemical solutions;
5. The construction of, and the filtering materials in, the roughing filters;
6. The method of operating the roughing filters, and the safeguards to prevent the deterioration of the effluent;
7. The control of the rate of filtration of the slow filters.

Among the novel features pertaining to the operation of the plant are the following:

1. The very high rate of operation of the slow filters;
2. The small quantity of coagulant necessary to secure satisfactory results—about one-fifth of that required in a mechanical filter plant of equal efficiency, and about one-eighth of that required to secure equally satisfactory results with coagulation and subsidence;
3. The ease of manipulation of the chemical dosing, and the certainty in its application;
4. The long runs of the roughing filters when no coagulant is necessary, and the consequent saving in wash-water, power, and attention;
5. The long runs of the slow filters, and the resulting large quantities of water filtered between scrapings;
6. The uniformity in the quality of the water applied to the slow filters, and consequently the satisfactory results obtained in the final purification;
7. The brilliancy of the effluent from the slow filters;
8. The absence of trouble with rusty hot water and troubles from the destruction of the solder in copper closet tanks;
9. The moderate cost of construction and operation.

The Borough of Steelton, having a population of approximately 15 000, is in Dauphin County, Pa., on the same side of the Susquehanna River as Harrisburg, and about 3 miles farther down. Topographically, the territory within the borough limits consists of a flat portion, about 20 ft. above the level of the river and from 1 000 to 1 500 ft. in width, which extends the whole length of the borough, and the residential district, which extends up the side hill to the east, and is much broken, and traversed by streets having for the

most part heavy grades. The principal street skirts the eastern edge of the flat area, the larger portion of which is occupied by the works of the Pennsylvania Steel Company.

Original Water-Works.—The original water-works plant was built by the Steelton Home Water Company, and was purchased by the borough in the latter part of 1899 for $150 069.48, since which time improvements to the intake, pumping station, and reservoir, and extensions of the street mains swelled the total cost to $223 000, on January 1st, 1906, of which $145 000 was the cost of the street mains, $38 000 the cost of the reservoir, and about $40 000 the cost of the intake and pumping station. The plant, at that time, obtained its water from the Susquehanna River through a 16-in. intake pipe reaching out approximately 1 400 ft. from the pumping station, to within about 150 ft. of the shore of Stuckers' Island. The original supply had been derived from wells dug on that island, but in a comparatively short time it became impossible to keep up the supply from the wells, and a pipe was extended about 50 ft. out into the river from the most southerly well, the water being then drawn from the river to the wells and back to the pumping station through the 16-in. pipe. Subsequently, owing to the silting up of the wells and their inaccessibility during high water in the river, a section of the 16-in pipe was taken out at a point about 150 ft. east of the island, allowing the water to enter the pipe from the river direct; it was in this condition on January 1st, 1907.

The water flowed through this broken-into intake pipe to a brick-and-concrete-lined well situated about 150 ft. from the river, just east of the Pennsylvania Railroad tracks, and was pumped from this well to the service reservoir, 270 ft. above the river, by two 1 500 000-gal. Deane, compound, direct-acting, outside-packed, plunger pumps, through a 12-in. force main. An additional 16-in. main, connected up at the pumping station to permit either pump to pump direct or to the reservoir, parallels the 12-in. force main for about 1 200 ft., extending to Second Street on Conestoga, and is cross-connected with the distribution system and the 12-in. force main. The service reservoir, when full, holds about 7 500 000 gal.

Character of Susquehanna River Water.—The Susquehanna River, above Steelton, has a water-shed of approximately 24 200 sq. miles. It drains practically one-half of the State of Pennsylvania, and about

6 080 sq. miles in the southern central portion of New York State. Upon its banks are many large cities, including Binghamton, Cooperstown, Corning, Elmira, Hornellsville, Horseheads, and Owego, in New York State, and Athens, Altoona, Bellefonte, Clearfield, Emporium, Harrisburg, Hazleton, Huntingdon, Lewistown, Lock Haven, Mahoning, Mifflintown, Mount Carmel, Nanticoke, Pittston, Plymouth, Sayre, Scranton, Shamokin, Sunbury, Towanda, Tunkhannock, Tyrone, Wilkes-Barre, and Williamsport, in Pennsylvania. Approximately, 1 000 000 persons reside in the cities and towns along the river and its branches above Steelton, of which probably 750 000 are located along the North Branch, about 100 000 on the West Branch, and 150 000 on the Juniata, which streams unite a few miles above Steelton to form the main Susquehanna River. In addition to this urban population, there is a very large rural population, and, of the larger cities, there are upwards of 60, having a total population of about 700 000, which have sewers discharging directly into the river and its branches.

The anthracite coal fields of Pennsylvania are on the water-shed of the North Branch, and along the whole length of the river and its tributaries there are many important manufacturing industries, the wastes from which are discharged into its waters. The West Branch lies largely in a sandstone country, and the principal industries along it have been in connection with the manufacture of lumber. The Juniata lies very largely in a limestone district.

In the days of canal navigation, several dams were built on each of the main streams, canals paralleling the streams on one side, or both, between the pools formed by the dams. The principal dams on the North Branch were at Clark's Ferry, Sunbury, Nanticoke, Wilkes-Barre, and Binghamton; on the West Branch, at Lewisburg, Muncy, Williamsport, Lock Haven, and Queen's Run; on the Juniata, at Millerstown, Lewistown, Newton, Hamilton, and Huntingdon; and on the branches at Pipers, Petersburg, Big Water Street, Little Water Street, Willow Street, Donnellys, Smokers, Mud, Williamsburg, Three Mile, Crooked, Frankstown, and Hollidaysburg. The existence of these dams is mentioned because they play an important part in the modification of the character of the river water during the season of low water as well as during the early stages of the fall and spring floods. Essentially, it will be seen that these various streams consist of a series

of comparatively steep natural slopes joining level pools, which, during low and moderate stages of water, form subsiding basins in which suspended matters carried by the water may be deposited in greater or lesser amounts. During freshets, however, the deposits which may have accumulated in the pools during the preceding quiet periods are scoured loose from the bottom, picked up by the rushing waters in a more or less changed condition, and carried down the stream in large quantities.

The larger part of the water-shed is semi-mountainous, the main streams flowing, for the greater part, through unerodable, and comparatively steep valleys, and the hillsides being largely denuded of forest areas; for this reason the stream and its branches are more or less flashy, big floods rising to a height of many feet in a few hours and falling almost as rapidly.

The amount and character of the sediment in a turbid water is of importance in determining the method of preliminary treatment best adapted to prepare the water for final purification. At Steelton, the quantity which will subside to the bottom of a 1-gal. bottle will sometimes reach a depth of ½ in., and is usually a composite of sand, coal dust, leaves, stones, silt, nitrogenous matter, coagulated clays, as well as aluminum hydrate and the hydrated oxides of iron mixed with various forms of dirt, bacteria, and other matters.

In the Susquehanna River water at Steelton a brown sediment, varying from red to yellow in color, either flocculent or otherwise (the flocculence depending largely on the stage of the river and the influence of the stage on the accumulation of bacterial masses and clay particles into agglomerations), is generally characteristic of the water in the west channel, although a sediment of this color may sometimes occur over the whole width of the river, following heavy general rains. A microscopic examination of this brownish sediment shows that it consists of particles of coal and dirt agglomerated by growths of small water plants and filamentous bacteria, probably of the order *Bacterium*, or *Leptothrix*.

A greenish-black sediment, resulting from the mixture of coal dust and clay, is generally flocculent, and is characteristic of the water in the east channel during heavy floods.

A grayish sediment, due either to finely-divided suspended matter other than clay, or to the hydrates of aluminum and of iron, which

settle to the bottom when the river is low, is characteristic of the water in the east channel, and is locally called "sulphur water."

A black sediment, generally more or less flocculent, consisting of finely-divided particles of coal from the mining districts, is characteristic of the east channel, and can be observed on the bottom of the river below islands and bridge piers, and in other protected positions when the river is at low stages.

All possible gradations of color intermediate between these are observable at different times owing to the commingling of the different waters.

The presence of red or yellow clay in the Susquehanna River water at Steelton, in increasing quantities, is almost uniformly accompanied by decreasing alkalinity; such water clears up slowly by subsidence. The water carrying flocculent sediment, which is generally black during floods but often the clean hydrate when the river is low, will clarify very quickly by subsidence. Such waters are characterized by low alkalinity and correspondingly high permanent hardness.

The color of the turbidity of the Susquehanna River water, therefore, is a fairly good index of its character, and, during the operation of the filters, plays an important rôle as a guide to the proper use of the coagulant.

The changes which take place in the character of the sediment, as the result of bacterial activity, are intimately associated with the iron, sulphur, and alkaline constituents of the water. Shortly after the Harrisburg experimental filtration plant was put in operation, in the fall of 1902, one of the roughing filters, then operating without a coagulant, became suddenly clogged to a condition of water-tightness within a very few minutes after the turbid raw water was turned on, and an examination disclosed a heavy blanket of black, sticky mud on the surface of the sand. It was surmised that this mud might be similar to that which was reported to have accumulated in the sedimentation basins at Cincinnati during the experiments conducted by George W. Fuller, M. Am. Soc. C. E., but a chemical examination of the mud on the Harrisburg filter showed that while iron was present in considerable quantities it was not in the sulphide form. The mud was found to consist essentially of very fine particles of coal dust mixed with the hydrates of iron and aluminum.

Although it had been understood for a long time that the drainage

from coal mines contained a considerable quantity of free sulphuric acid, an explanation of the processes by which this was formed was not generally current. Recognizing that ferrous sulphide, as pyrites, was present in the coal dust in the river, it was suspected that the "sulphur bacteria" and "iron bacteria," which work various transformations in iron compounds, might play some part in the formation of this sulphuric acid, and an experiment was made to test the hypothesis. A small quantity of coal dust or culm was pulverized in a mortar, and equal portions were placed in narrow-mouthed bottles. In each of these bottles was then placed a definite quantity of untreated river water, and, the mouths of the bottles having been closed with cotton plugs, half the number were sterilized in an autoclave at a pressure of 15 lb. for 30 min. After standing in the laboratory for about a month the bottles which had not been sterilized exhibited very different characteristics from those which had been so treated. After both sets of bottles had been shaken violently the turbidity in the set which had not been sterilized was observed to decrease very much more rapidly than that in the set which had been sterilized, somewhat after the manner that the untreated river water containing this hydrated oxide of iron acts in comparison with water which does not contain it. The water in these bottles represented, in some degree, the condition of the water in the river, except that the bottles which had been sterilized contained no living bacteria, while the others, supposedly, did, having been filled with river water which at the time of filling was known to contain a large number. The results of this preliminary experiment seemed to indicate that the change which had taken place in the unsterlized bottles was due to the action of the bacteria on the iron compounds contained in the culm.

Following out this idea, a further elaborate set of tests finally led to the conviction that a hydrate of iron was formed from the pyrites, in the presence of bacteria, while it was not formed when bacteria were absent; further, there was no evidence of the formation of soluble iron except when bacteria were present.

The reactions which took place in the bottles used in the experiments are not known, and are probably of a very complicated character, some resulting directly from, or being intimately associated with, the biological activities present, and others in all probability being spontaneous chemical reactions. It would be foreign to the subject of

this paper to enter into a discussion of these experiments in detail and of others which have since been made both on Susquehanna and on Schuylkill waters; it will be sufficient to state that at the conclusion of the Harrisburg experiments the data indicated that the bacteria occurring normally in the Susquehanna River, in the presence of organic matter on which they can live, and under proper conditions of storage, are able to transform the pyrites in the coal into compounds which furnish the material for the hydrated oxide of iron naturally occurring in the water. The indications seem to be that the sulphur in the pyrites, in contact with water, furnishes the material from which the bacteria make sulphuric acid, which, coming in contact with the iron, forms the soluble ferrous sulphate. The water containing this soluble ferrous sulphate commingles with the alkaline waters of the river farther down stream, the sulphate of iron being decomposed, the sulphur combining with the lime and magnesia to form the sulphates of these bases, and the iron being converted into the hydrate and subsequently into the hydrated oxide. This seems to be at least one method by which free sulphuric acid is formed in the drainage of the mining regions; the process also takes place to a very considerable extent in the level reaches of the river, where culm is deposited as floods recede. When the river rises, and the dirt on these flats is washed away, there is borne with it the hydrate and hydrated oxide of iron which had been formed during exposure. At the Clark's Ferry dam, some miles above Harrisburg, the water in the east channel sometimes shows this coagulated condition so plainly that frequently within an area of a few square feet one may collect a sample of either perfectly clear, or very black water.

At times of very low water there occurs along the east shore a brownish flocculent sediment composed of the hydrate of aluminum as well as the hydrate of iron. The source of the aluminum entering into the composition of this hydrate is unknown; possibly it is derived in some way from the clays.

As a result of the natural formation of the hydrated oxide of iron, and of aluminum hydrate, in the water of the river, a considerable degree of apparent self-purification takes place during the summer months. This is brought about, in the pools above the dams, by the entangling of the bacteria into flocculent masses and their consequent settlement to the bottom of the river with the other suspended

matter. The action is, on a large scale, very similar to the results of the treatment of sewage by chemical precipitation, and the active precipitants are the same as those used in the artificial process. Ultimately, a large proportion of the organisms thus carried to the bottom cease to exist in an active condition, finding their environment not suited to sustain their vitality. The apparent purification is at times so great that, although from the many cities along its course enough sewage is flowing continuously into the river to render the water at its extreme low stage offensive, yet at such times the water, as it flows past Steelton, is actually at its best, both in appearance and as to its hygienic qualities. The analyses of different samples of river water collected at Harrisburg during low water have shown as few as 10 colonies of bacteria per cu. cm., with no intestinal bacteria present; and generally, during low-water stages, when the quantity of sewage discharged into the river is greatest relatively to the stream flow, the numbers of bacteria in the river water opposite Steelton (from points far enough away to be beyond the influence of the Harrisburg sewage) are lowest and the numbers of intestinal bacteria fewest. This condition obtains, however, only during dry weather, for the first floods which come down in the fall scour out a portion of the deposits from the pools formed by the dams up stream, and then the counts show sometimes as many as 300 000 colonies of bacteria per cu. cm. of water, as well as large numbers of bacteria characteristic of intestinal discharges.

The total hardness of the river water varies with the stage of the river, yet not necessarily in proportion thereto, and is about the same, as a general thing, from bank to bank. The water is hardest at low stages when the flow is largely from springs, that on the west side being high in carbonates and low in sulphates while that on the east side is low in carbonates and high in sulphates, owing to the chemical changes taking place, as above described.

These modifying conditions destroy any relationship between turbidities, river stages, and numbers of bacteria. High bacterial counts sometimes accompany very low, as well as very high, turbidities, and, similarly, relatively low turbidities are sometimes found at very high stages of the river; further, the color of the turbidity is no indication of its total amount, as a comparison of the records of ten samples of water, each with a turbidity of 1 000 parts per million, collected at

different times, and at different parts of the river, shows all gradations of colors between bright yellow, red, dark brown, and black.

As a result of these complex conditions, the water in the river may change quickly from a moderately hard, clear water to a soft water carrying several thousand parts per million of turbidity, which, on occasions, may intermittently come down coagulated to a more or less slimy or flocculent condition with hydrated oxide of iron and hydrate of alumina. On the other hand, the river from bank to bank may be carrying its maximum load of yellow clay from the Conodoguinet and Juniata, or reddish-brown mud from the West Branch. During a general flood from the entire water-shed, all these conditions may prevail at the same time, the yellow water from the Juniata keeping on the western side, the West Branch water keeping in the central part, and the black water from the North Branch along the eastern shore, each stream occupying practically one-third the width of the river, which, opposite Steelton, is approximately four-fifths of a mile.

Although of great width, the river is comparatively shallow, the bed in that vicinity being of shale and limestone, and the depth of the water at its lowest stage being not more than 3 ft. at any point and not more than 1½ ft. for the greater part of the width. Some trouble is caused during the winter by slush-ice and anchor-ice, the formation of which is favored by these conditions.

The Steelton Filter Plant.

Those who have had experience in the filtration of polluted waters will appreciate some of the difficulties encountered in the purification of the Susquehanna River water, particularly as to bacterial purification, for the reason that the numbers of bacteria in the raw water may be increasing rapidly while the turbidity is decreasing; and hence, if the attempt is made to control the bacterial purification by operating the plant on the indications of the removal of turbidity only, the result may be anything but satisfactory.

Another complication arises from the exhaustion of the alkalinity of the raw water by the acid mine wastes, and also its reduction by dilution during heavy freshets. This, however, can readily be taken care of by adding enough lime or soda-ash to supply the deficiency in alkalinity, in case a coagulant is used in connection with the process

of purification. Among other complications is the difficulty of securing the satisfactory coagulation of the raw water when the bacterial counts are high and the turbidity is low, and also the difficulties encountered in securing a sufficiently heavy coagulum when the temperature of the water approaches the freezing point, at which times the turbidities are also usually low.

While all these difficulties can be overcome and the water can be handled successfully with a mechanical filter plant such as that installed for the City of Harrisburg, the successful operation of such a plant depends on the maintenance of a fully equipped laboratory, and on a superior grade of supervision; and, to secure these, it is essential, of course, that the revenues derived from the sale of the filtered water be sufficient to cover the additional expense.

Owing to the considerable general pollution of the river, the discharge of the sewage of Harrisburg into it only a couple of miles above Steelton, the excessive turbidity, and the enormous quantities of sand and coal delivered into the borough's distributing reservoir by the supply pumps (ultimately to enter the distribution mains, interfering with the action of meters, and clogging service pipes), the question of securing a purer and more satisfactory source of supply was brought to an issue in the winter of 1906, and the writer was commissioned to investigate and report on what could be done toward securing a new supply of a satisfactory quality, or toward purifying the existing supply. The report recommending the purification of the present supply was submitted to Councils early in 1907, and, following its adoption, the borough voted to issue $80 000 in bonds to cover the cost of the necessary works.

The plant, as built, was designed to place the securing of satisfactory results, as to purification, within the reach of the Water Department without the necessity of maintaining a laboratory, and without incurring unduly heavy construction and operation costs. The general basis of the design is as follows:

First.—The removal by subsidence of such of the suspended matters as will settle out from the raw water in about 12 min.;

Second.—The removal, by passing the water rapidly through deep, coarse-grained filters, of at least 90% of the applied turbidity, using a coagulant to secure this result when necessary, and allowing no coagulated turbidity greater than 25 parts per million to issue from

the coarse-grained filters, no matter what the turbidity of the applied water;

Third.—The filtration of the effluent of the coarse-grained filters through slow sand filters at a relatively high rate.

New Intake.

The works include a new 30-in. cast-iron intake pipe extending out about 1 500 ft. into the river to a point where observations indicated that there would be a minimum of trouble with silt, sand, and coal dust. At the point where the intake was located prior to the commencement of these improvements large quantities of sand and fine particles of coal were drawn into the intake well during floods in the river, as much as a car load a week requiring removal in order to keep the pumps in operation.

The new intake pipe was laid to proper line and grade, in cofferdams, in a trench excavated in the rock bottom of the river, the joints being poured with lead and caulked water-tight.

The intake cage consists of a reinforced concrete structure inclosing the end of the intake pipe and spreading out like a fan to give an opening in front 1 ft. high and 20 ft. long, with vertical screen bars of 1-in. round steel, spaced 6 in. apart, in two rows, and staggered. The top of the opening was located at a depth of 1 ft. below extreme low water, and the rock bottom of the river was blasted away in front of the cage, a smooth concrete apron being laid thereon to assist the current in rolling sand and coal particles along past the intake. The line of the face of the intake is parallel with the trend of the current of the river at that point.

A valve is placed in the intake pipe, at the shore end, and a connection is made with the existing 24-in. pipe leading to the suction well. The work on the intake, which was commenced on July 1st and completed early in September, 1908, was done by the Water Board by day labor, from the plans and under the general supervision of the writer.

Improvements at the Pumping Station.

New Pumps.—The plans, as approved by Councils on July 10th, 1907, called for the construction of a 3 000 000-gal. slow sand filter plant with roughing filters, to be built on a piece of ground 100 ft. wide and 265 ft. long, situated about 700 ft. east and 900 ft. north of

the pumping station. Before the construction of the works had been authorized, appropriations had been made for the installation of a new 3 000 000-gal., Heissler, compound-condensing, crank-and-fly-wheel, pumping engine at the pumping station.

In order to suit the new conditions, it was necessary to remodel the piping in the pumping station and install two 12-in. centrifugal pumps, direct-connected to simple, vertical, condensing engines of the marine type, the suction pipes of the new pumps being connected to the remodeled original suction to which the suctions of the old Deane pumps were attached. The discharge pipe from the two centrifugal pumps was piped to the filter plant, with provision for a cross-connection to the suction pipe of the new Heissler pump, so that, in case of accident to the new filtered-water well, raw water could be pumped directly into the Heissler pump and thence to the city. The Deane pumps were located in a pit sufficiently low to enable them originally to take their own suction from the intake well, but the new Heissler pump, as well as the remodeled Deane pumps now take the filtered water from a new filtered-water well constructed just outside of the pumping station.

From this brief description it will be seen that in the remodeled plant the raw water is pumped from the intake well by centrifugal pumps to the filter plant, and the filtered water is returned to the new pump well just outside of the pumping station, from which either the new Heissler or the old Deane pumps can draw the filtered water and force it through a 12-in. force main to the distribution reservoir on the hill above the town, or through the 16-in. return main directly into the distribution system, or into the reservoir and the distribution system at the same time by properly manipulating the control valves on the cross-connections. By this arrangement the construction of an additional force main to the reservoir, which would have been an immediate necessity owing to the increased consumption of water, was deferred for many years by reason of the provision of a pure water which could be pumped into the mains direct.

Raw-Water Delivery and Filtered-Water Return Mains.—The raw-water pipe leading from the pumping station to the filter plant, and the return pipe leading from the filter plant to the new pump-well are of wood-stave construction, with cast-iron specials for bends and connections to the concrete.

The staves were of Oregon pine dressed to the true circle and constructed in the usual manner with steel tongues in the end joints of abutting staves, and ½-in. round steel bands, with rolled threads and standard hexagonal nuts engaging against cast malleable-iron shoes, the bands being placed 12 in. apart, excepting at butt-joints and on the ends of cast-iron specials, where the spacing was reduced to about 4 in. The shoes and bands were dipped in a hot bath of Pioneer mineral-rubber pipe coating, and were retouched, in the trench, after cinching, with Smith's Durable Metal Coating applied with a brush.

The raw-water and filtered-water pipes have an internal diameter of 24 in., and, for the greater part of their length, are laid in one trench.

Filtered-Water Well.—The new pump well at the pumping station is of concrete. It is circular in plan, has an internal diameter of 15 ft., and a depth of 14 ft. from the floor to the underside of the reinforced roof. It has a cast-iron manhole frame and cover, the opening being 36 in. in diameter; the cover is provided with a gasket and is bolted down water-tight to the frame, which is built into the concrete.

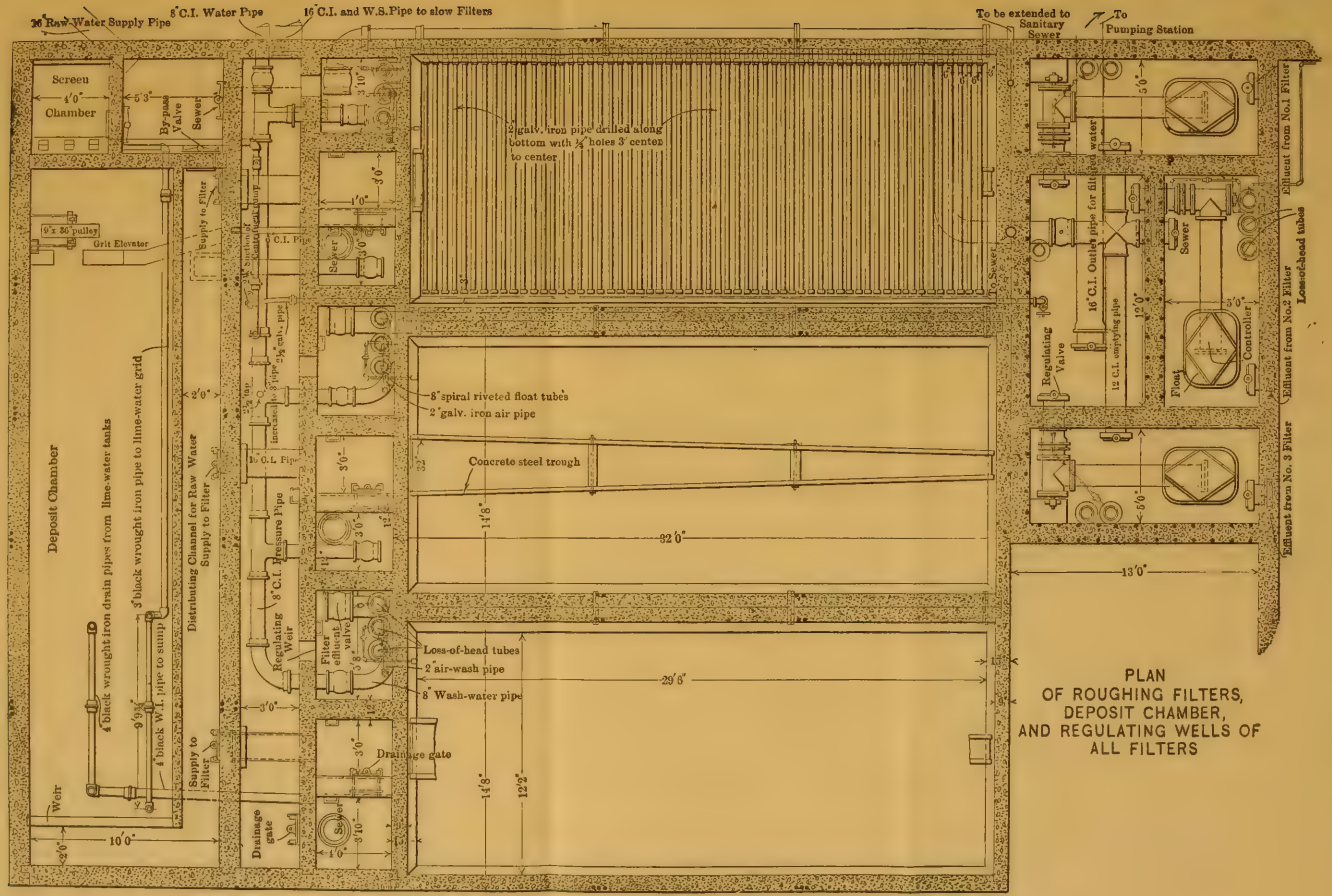
Where the 14-in. discharge pipe from the centrifugal pumps passes through this well to form the connection with the wood-stave pipe, a by-pass connection is made by which the raw water can be discharged into the filtered-water well in case of necessity, but the valves and pipes are arranged so that any leakage in or through the valve on the raw-water pipe will drain out to the raw-water well and not contaminate the filtered water.

Purification Works.

Capacity of Plant.—The purification works include a deposit chamber and three roughing filters which, with their regulating wells and the regulating wells for the slow filters, occupy a space approximately 65 ft. long and 44 ft. wide; in addition there are three slow filters which occupy the remaining available space, 100 ft. wide and 200 ft. long.

The plant is designed to supply 3 000 000 gal. of filtered water per day, a proper reserve capacity being provided to permit of cleaning the filters while delivering the stated quantity of water.

Method of Purification.—The method of purification consists in



PLAN
OF ROUGHING FILTERS,
DEPOSIT CHAMBER,
AND REGULATING WELLS OF
ALL FILTERS

preparing the raw water for slow filtration by removing therefrom enough of the turbidity and bacteria to permit the slow filters to be operated at a relatively high rate. The object of the treatment is to prepare the water in such a way as to be able to discharge upon the slow filters, for final purification, a water of reasonably constant composition, as to turbidity and bacterial contents, throughout the entire year, keeping the maximum limits of turbidity and numbers of bacteria in the applied water down below figures which experience indicates to be desirable.

The treatment begins with the removal of the floating matters from the water by coarse screening and of the particles of sand, silt, and coal dust by subsidence. The water is then passed through roughing filters containing a deep bed of coarse-grained filtering material in which an additional portion of the remaining suspended matter (including a varying percentage of bacteria) is retained, the effluent water passing to the slow sand filters for final purification. As a general rule, when the numbers of bacteria in the raw water are relatively low and when the turbidity does not exceed 50 parts per million, the roughing filters produce an effluent of satisfactory quality without the use of artificial coagulation; but when the turbidity of the raw water is higher than 50 parts per million, or when it is caused by particularly fine particles, or when the bacteria number more than about 5 000 per cu. cm. in the raw water, a sufficient quantity of coagulant is mixed with the incoming raw water, as it enters the deposit chamber, to produce a rough filtered effluent with a turbidity of not more than 10% of that of the applied water, and not more than 25 parts per million, in any case. Adherence to these limits, which are as yet only tentative, will apparently give sufficiently long runs of the roughing filters to permit of easily keeping the plant in operation, and sufficiently long periods of operation of the slow filters to enable them to pass a satisfactory quantity of filtered water between scrapings.

The period of subsidence allowed in the deposit chamber, for the maximum capacity for which the plant is designed, is 12 min. The roughing filters are designed to operate at a rate of 172 000 000 gal. per acre of filter surface per day when delivering 1 500 000 gal. per filter, three filters being provided so that two can yield the full 3 000 000 gal. while the third is being washed. The net area of the sand surface of each slow filter is 0.1446 acre, and when delivering 1 500 000 gal.

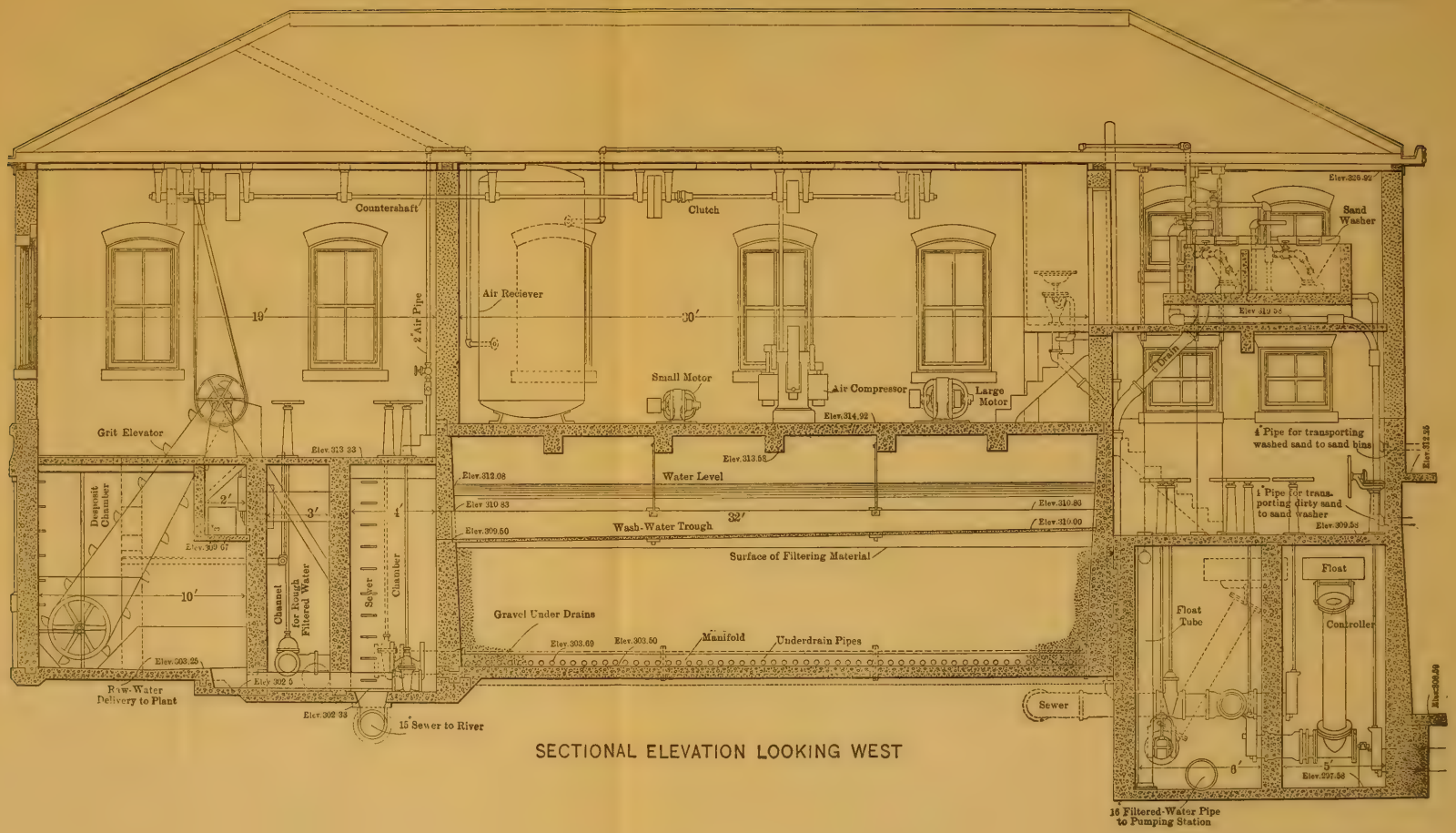
per day the filter will operate at a rate of 10 373 000 gal. per acre per day, two filters yielding 3 000 000 gal. daily with the third in reserve for cleaning.

Deposit Chamber.—The raw water is received at the filter plant at the bottom of a rectangular compartment in one end of the reinforced concrete deposit chamber, Plate XLVII, a valve-controlled connection being provided between the bottom of this compartment and the sewer. One outlet from the upper part of this compartment leads to the deposit chamber and a second leads directly to the channel feeding the water to the roughing filters. By using one or the other of these outlets the water may be passed through the deposit chamber, or, if desired, the deposit chamber may be by-passed and the raw water delivered directly to the roughing filters. After leaving the inlet chamber, on its way to the deposit chamber, the water first passes vertically downward through a coarse screen composed of parallel lines of 2-in. planks standing on edge, horizontally, and spaced about 8 in. apart from center to center transversely to the direction of the entering water; between these planks, and in a vertical plane, one above the other, are the horizontal perforated pipes through which the lime-water and coagulant solutions are admitted to the raw water.

The raw water is admitted to the deposit chamber in the manner described, in order to use up the entering velocity head and permit the water to pass through the deposit chamber quietly and thus drop out as much as possible of the suspended matter in the short time allowed therefor.

Grit Elevator.—Near the end at which the raw water enters, a grit elevator is installed to remove continuously the sand and coal particles which may collect in the deposit chamber. The elevator, Plates XLVII and XLVIII, which has an estimated capacity of 20 tons per 24 hours, consists of an 8-in., 6-ply rubber belt carrying perforated, galvanized-iron buckets, 6 in. long, 4 in. wide and 4 in. deep, at intervals of 2 ft. The belt travels at the rate of 25 ft. per min., and the buckets discharge into a hopper arranged to fit into the end of a 6-in. cast-iron pipe leading to one of the sewer compartments of a roughing filter.

The deposit chamber is 10 ft. wide and 42 ft. long, including the entrance chamber and screen box, and at full capacity the water flows through it with a depth of about 9.2 ft. and a velocity of 0.05 ft.



SECTIONAL ELEVATION LOOKING WEST

per sec., passing over a weir at the farther end into a channel leading to the inlet openings to the roughing filters.

Chemical-Mixing Tanks. Vertically above the deposit chamber, and carried on heavy reinforced concrete beams spanning the chamber, stand the mixing tanks, Plates XLIX and L, for the lime-water and coagulant solutions. These tanks are of monolithic reinforced concrete. In each tank there is a wooden dissolving rack with an open bottom carrying wooden frames provided with copper screens having 20 meshes per inch, reinforced by a coarse copper screen having 4 meshes per inch, with No. 11 bars. Each tank is provided with a system of $\frac{3}{4}$-in. perforated copper pipes at the bottom connecting with a line of galvanized-iron pipe leading from above the tops of the tanks to the air receiver, to provide for the agitation of the solution after it has been mixed. A galvanized-iron steam connection, with a copper branch reaching to the bottom of the coagulant tanks, is provided for warming the solution during cold weather. The solution is mixed in the usual manner. With a 2% solution, two tanksful will furnish a dose of 2 gr. per gal. for a consumption of 3 000 000 gal. daily. The coagulant solution is drawn from the mixing tanks through 2-in. copper pipes arranged with swivel-joints to permit of decanting the solution at any desired depth.

Coagulant-Measuring Box.— Through the copper outlet pipes the solution runs to the coagulant-measuring box, Fig. 1, and Fig. 1, Plate LI. This is a reinforced concrete box with a plate-glass front in which are drilled three $\frac{1}{4}$-in. and three $\frac{1}{2}$-in. orifices, all at the same elevation and spaced 4 diameters apart and 4 diameters of the $\frac{1}{2}$-in. holes from the bottom and side edges of the plate. This glass plate is 11 in. wide, $\frac{1}{4}$ in. thick, and 28 in. high, and the holes are countersunk one-half the depth of the plate from the outside. On the back of the plate is etched a graduated scale reading to hundredths of a foot for a height of 2 ft. above the centers of the orifices. The plate is fastened to the front of the orifice box on soft rubber gaskets, made of tubing, by brass plates and expansion bolts secured in the concrete. The depth of the solution in the orifice box is regulated with a bronze lever valve and float arranged to admit the solution near the bottom and permit the regulation of the depth in the box by changing the height of the float. The float is of copper, 10 in. in diameter, with a flat bottom and a coned top, sliding on a $\frac{1}{4}$-in. brass rod, the rod being

hinged to the lever of the valve, and the height of the float being adjusted on the rod with a bronze set screw having a milled head.

Each orifice is provided with a soft rubber cork, or stopper, attached to a handle consisting of a piece of brass pipe with a cap on one end and an elbow on the other, the cork being fastened to a nipple entering the elbow; there is a rack, with thumb-screws, across the top of the orifice box, for holding the handles where desired.

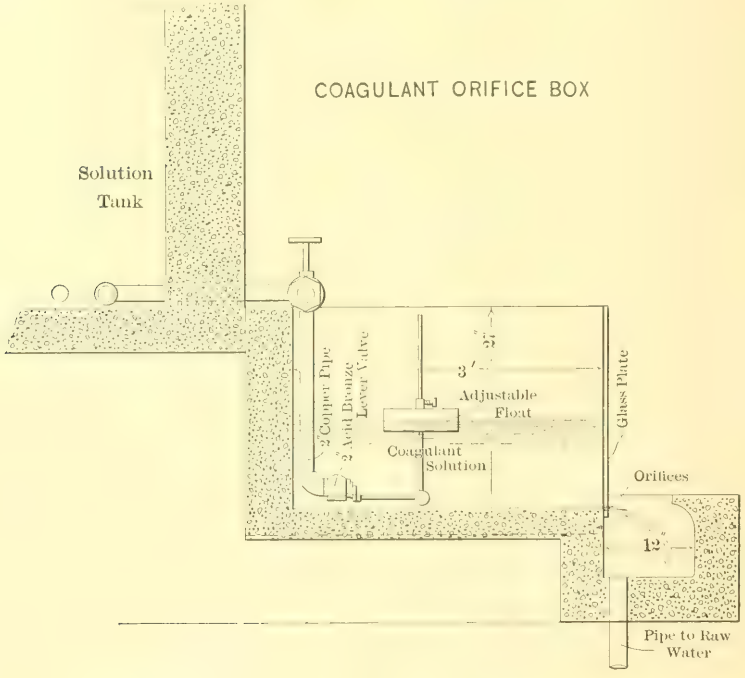
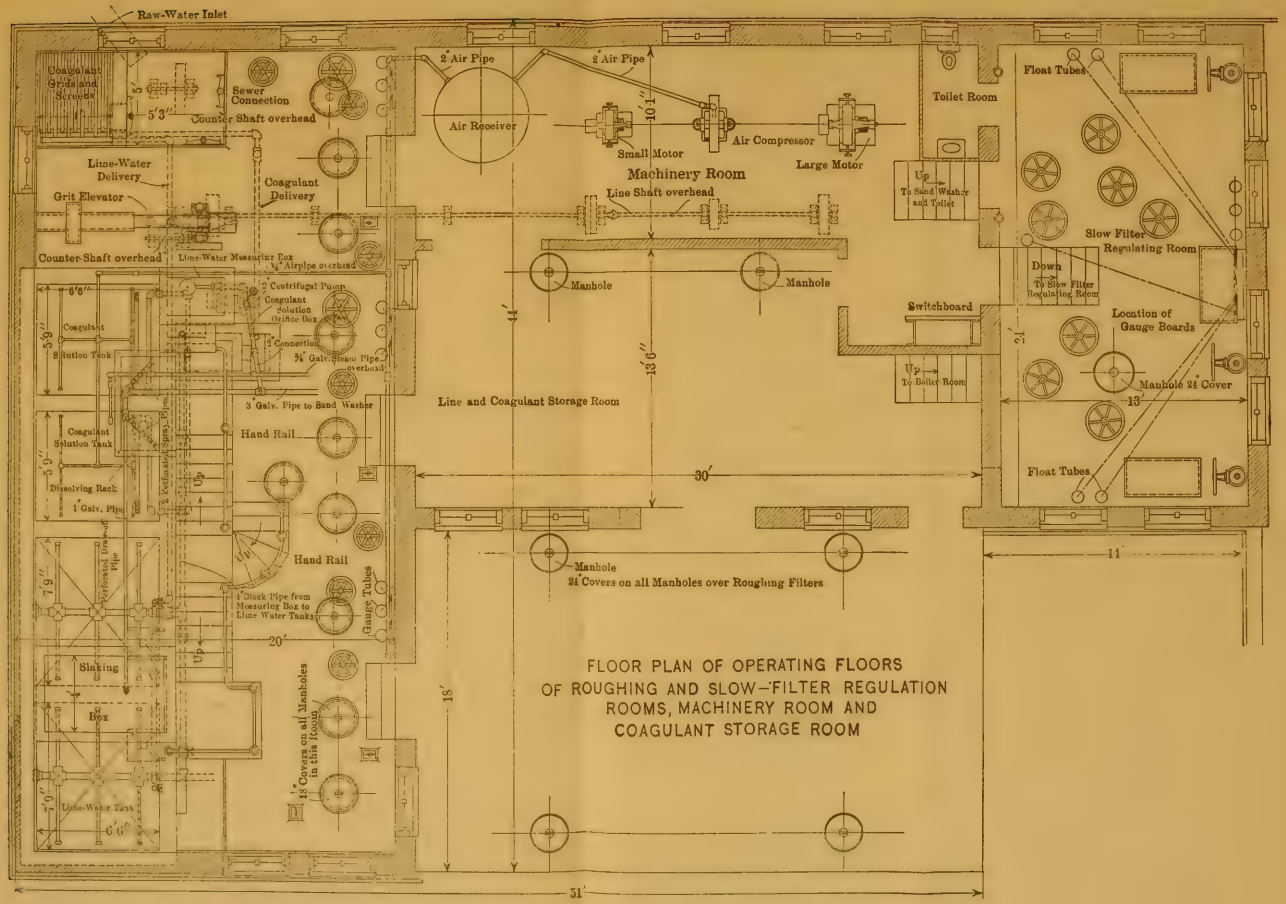


FIG. 1.

The coagulant solution flows through the orifices in the glass plate to a small concrete box, from which a 2-in. copper pipe connects with the perforated pipes between the screen bars at the entrance to the deposit chamber.

Lime-Water Mixing Tanks.—The lime-water apparatus consists of two tanks for making the lime-water (Plates XLIX and L), a slacking box, and an orifice box for measuring the quantity of water to be supplied to the lime-water tanks. Lime being only slightly soluble in water, a relatively large quantity of solution is required



FLOOR PLAN OF OPERATING FLOORS OF ROUGHING AND SLOW-FILTER REGULATION ROOMS, MACHINERY ROOM AND COAGULANT STORAGE ROOM

unless the lime be added as cream of lime. In the Steelton plant the apparatus is designed to make use of a saturated solution of lime-water.

The lime is slaked in a reinforced concrete box supported on top of the partition separating the two lime-mixing tanks, a sluice-gate being provided on each side for emptying the milk of lime through funnels into pipes leading down to the bottom of the lime-mixing tanks. The latter are rectangular in plan, the bottom of each being in the form of an inverted frustum of a pyramid. From the bottom of each a pipe leads to the sewer, with a connection immediately below the bottom of each tank leading to the lime-water measuring box which stands on brackets above the coagulant-mixing tanks and receives its water through a float and lever valve (with spindle and float as described for the coagulant orifice box) either from the city mains or from a 2-in. centrifugal pump taking its suction from the channel containing the rough-filtered water. Valves in the various pipes permit of cleaning the tanks and pipes, and the admission of water to either of the tanks as desired. The lime-water orifice box is also of reinforced concrete, with a plate-glass front, $\frac{1}{2}$ in. thick, 20 in. wide and 28 in. long, containing two $\frac{1}{2}$-in., two 1-in., and one $1\frac{1}{2}$-in., orifices; there is also a graduated scale etched on the back of the glass, the details of the arrangements being similar to those for the coagulant orifice box. The effluent from the solution-measuring box issues vertically into the mixing tanks through the center of the bottom, rising as a saturated solution through the milk of lime, which has been mixed and deposited in the bottom of the tank. The quantity of water rising through the tank is varied from time to time in accordance with the different conditions of the raw water and the varying consumption of water by the borough, the maximum rate, for a dose of 2 gr. per gal., for a consumption of 3 000 000 gal. daily with both tanks in service, being such that the water would rise through the tanks at the rate of 7 in. per hour if the outlet valves were closed. The water, having picked up its lime at the bottom of the tank, is skimmed off just below the surface by a system of perforated, horizontal, galvanized-iron pipes.

Application of Chemicals.— For mixing the lime-water with the raw water, six lines of $1\frac{1}{2}$-in. galvanized-iron pipes, perforated with double rows of $\frac{1}{4}$-in. holes $3\frac{1}{2}$ in. apart, screwed into a 4-in. galvanized-iron manifold connected with a loose-sleeve joint to the lime-water

supply pipe, lie between the wooden bars of the screen at the entrance to the deposit chamber. The copper pipes for distributing the coagulant solution consist similarly of six lines of 1-in. pipes perforated with double rows of $\frac{3}{16}$-in. holes spaced $3\frac{3}{8}$ in. apart and fastened to a $2\frac{1}{2}$-in. copper manifold; they lie vertically below the lime-water mixing pipes.

Roughing Filters.

The raw water, after passing through the deposit chamber, or in case it is by-passed around the deposit chamber, enters a channel, running lengthwise along one side of the deposit chamber from the

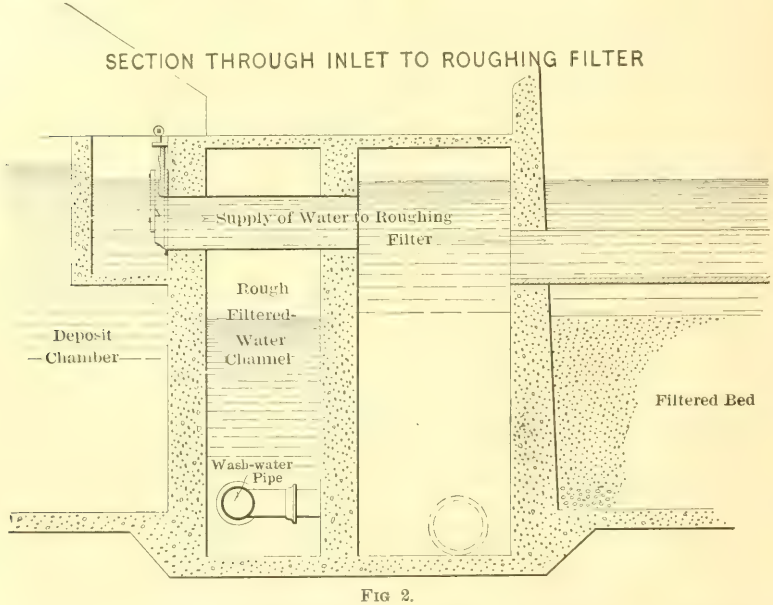
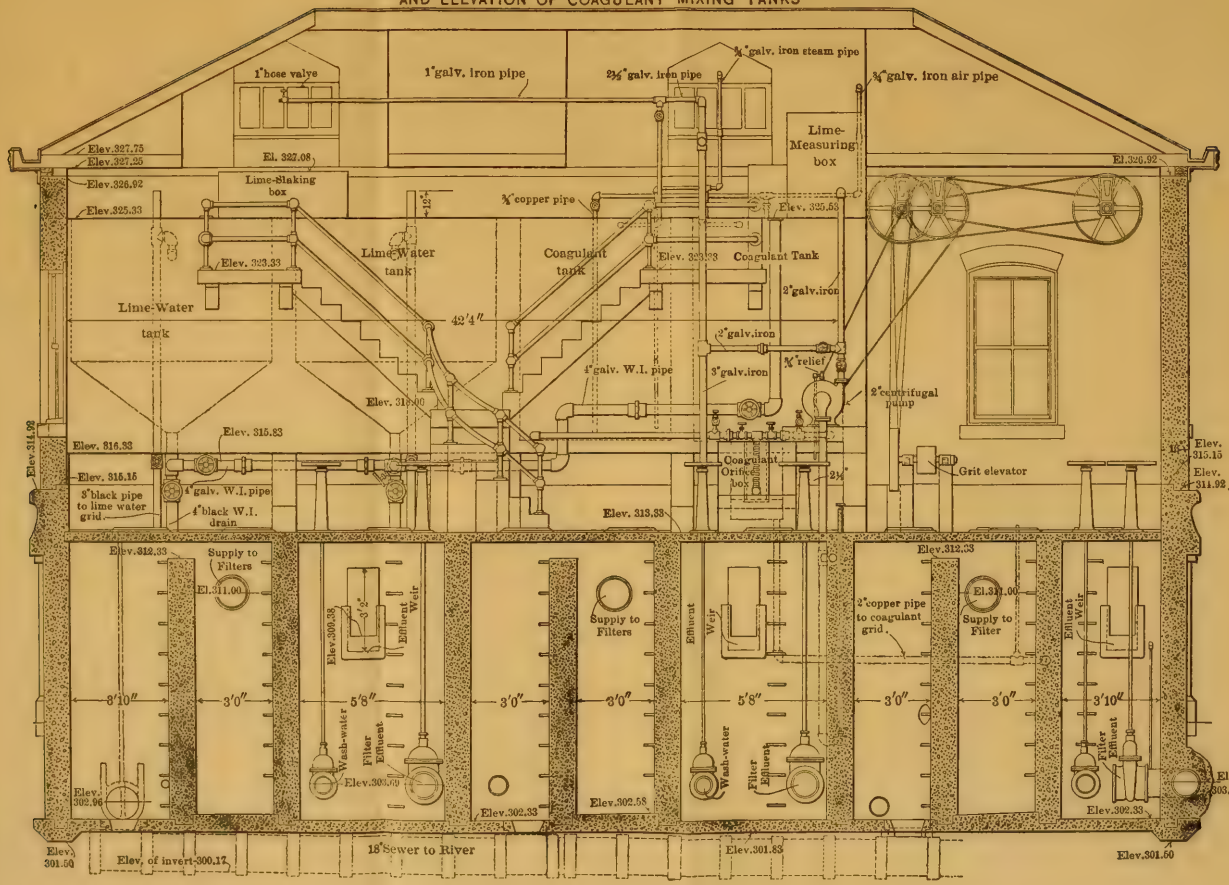


Fig 2.

weir at the east end to the entrance chamber at the west end, the side and bottom of this channel being supported by vertical steel bars built in the concrete and passing continuously through the side of the channel and the beams spanning the deposit chamber and carrying the mixing tanks. From this channel, Plate XLVII, the water is taken to each of the three roughing filters through 16-in. cast-iron pipes controlled by 16-in. sluice-gates operated with chain-hoists. Between the ends of the three filters and the east side of the deposit chamber are arranged a series of chambers, three for each filter, containing connections to the sewer, to the filtered-water effluent pipe, and to the raw-water supply, a connection being made at the bottom, between this

SECTION THROUGH REGULATING WELLS OF ROUGHING FILTERS, AND ELEVATION OF COAGULANT MIXING TANKS



latter chamber and the chamber leading to the sewer, the purpose of which is to permit the dirty water resulting from the washing of the filters to flow backward through this inlet chamber to the sewer leading to the river. Between the row of regulating chambers for the filters and the deposit chamber is a narrow channel, 3 ft. wide, and about 10.5 ft. deep, into which the rough-filtered water is discharged from the roughing filters, and from which the supply pipe leads to the slow filters.

The 16-in. supply pipes to the roughing filters cross the channel containing the rough-filtered water, and enter the supply chambers immediately in front of the center of each roughing filter. The wash-water troughs, which serve the purpose of distributing the incoming water over the filters during operation, as well as of removing the wash-water when the filters are being cleaned, are suspended from the roof beams of the roughing filters by 1-in. bolts engaging saddle pieces of flat steel, over the top and under the bottom of the troughs, with jamb-nuts at top and bottom to permit of adjustment.

The roughing filters are each 12 ft. 2 in. wide and 29 ft. 6 in. long at the bottom, the sides and ends battering outward 6 in. in the height of the filter, the outside surfaces of the walls being vertical. The side walls and roofs of the filters are of reinforced concrete, and the bottoms of concrete without reinforcement.

Roughing-Filter Underdrains.—In each filter there is an underdrainage system, similar in principle to that designed by the writer in 1902 for the Harrisburg Filter Plant. It consists of 59 parallel 2-in. galvanized-iron pipes, extending across the width of the filter and drilled along the bottom with ¼-in. holes 3 in from center to center, the pipes entering a manifold made of 12-in. flanged pipe built into the concrete wall on one side of the filter and terminating in the effluent regulating chamber in front of the filter in a 12-in. gate-valve operated by hand from the floor above. The 2-in. galvanized-iron pipes are placed in the filter so that their bottoms stand 1 in. above the floor of the filter, and are surrounded with a 4-in. layer of gravel composed of stones which will pass through a screen having ¾-in. meshes in the clear between the wires and remain on a screen having ¼-in meshes in the clear between the wires, and containing no particles finer than ¼ in. in largest dimensions. Upon this rests a second layer of fine gravel, 3 in. thick, all the particles of which will pass through a screen having ¼-in. meshes, but will remain on a standard brass sieve having

12 meshes per lin. in., and containing not more than 2% of particles which will pass through a standard sieve having 15 meshes per lin. in. These specifications were written for local materials, and might require modification for other materials of different subsiding values.

Filtering Materials in Roughing Filters.—The filtering material in the roughing filters consists of a layer of fine anthracite coal screenings, 5 ft. thick, prepared from particles of fine coal washed down the river from the culm piles of the mines on the North Branch water-shed during floods and recovered from the river in the neighborhood of Harrisburg and Steelton by centrifugal pumps. Considerable care was required in the selection of the raw material, as it was desired that the finished product should have an effective size of about 1 mm. and a uniformity coefficient as low as practicable without making the cost of preparation too great. The material as received at the plant contained considerable moisture in the pile and was prepared by casting over a screen having 4 meshes per in., the actual separation with the wet coal being practically the same as would be obtained by screening the dry material through a sieve having 6 meshes per in. Each of the three filters contains 5 ft. in depth of this screened coal, which, as prepared, has an effective size of 1 mm., a uniformity coefficient of about 2.4, and does not contain more than 2% of particles which would pass through a screen having 30 meshes per in. In preparing the river coal, 144 tons of the finished product, weighing about 53 lb. per cu. ft., was required, to secure which it was necessary to screen 176 tons, rejecting 32 tons of coarse particles.

Roughing-Filter Controller.—The rate of filtration of the roughing filters is controlled by regulating the depth at which the rough-filtered water flows over the edge of standard bronze weir-plates, 12 in. long, Plate L and Fig. 3, the weirs being placed in orifices in the front walls of the effluent regulating chambers of the filters, the rough-filtered water falling over the weirs and into its channel, from which it is conducted to the slow filters, the elevation of the weirs being fixed so as to allow losses of head up to about 2½ ft.

Loss-of-Head and Rate Gauges.—The rate of filtration and loss of head are indicated, for each filter, by gauges and indexes operated by spherical, seamless, spun-copper floats, resting on the water in three 8-in. spiral riveted tubes, Plate XLVII and Fig. 3, placed in the regulating chamber of each roughing filter.

FIG. 1.—ROUGHING-FILTER OPERATING ROOM.

FIG. 2.—FORMS FOR ROOFS OF SLOW FILTER.

The rate gauge was graduated at the plant from the actual measured discharge over each weir in stated periods of time, and at different depths of flow, as indicated by the float gauge. The loss-of-head gauge was divided into feet and hundredths.

The gauge boards were made of strips of poplar, painted with three coats of white-zinc paint, graduated by hand with indelible India ink, the figures and letters being lead pattern-makers' letters attached to the gauge boards with shellac, on the first coat of paint, and painted white with the board before the latter was graduated. After graduation the tops of the letters were blackened, and the board was given two coats of spar varnish and stiffened by small brass cleats

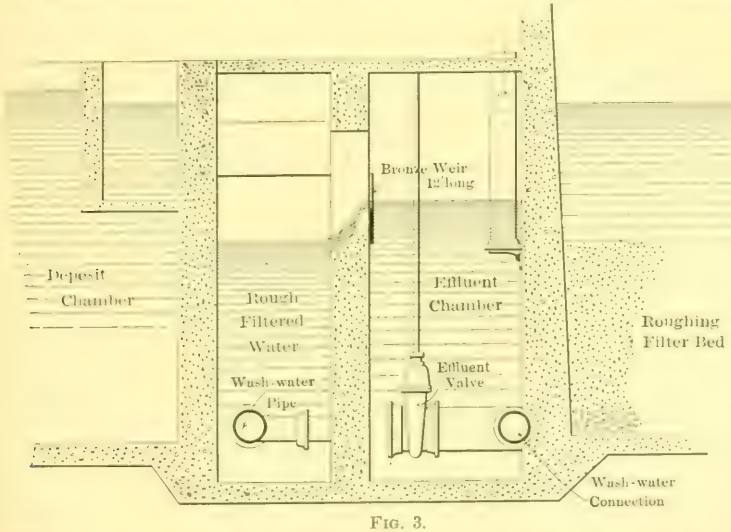


FIG. 3.

screwed to the backs at the top and bottom, the cleats being provided with ears on each side drilled with holes, $\frac{1}{2}$ in. in diameter, parallel to the back of the board and lengthwise of the same. The gauges were strung on parallel guides formed of No. 20, B. & S. gauge, phosphor-bronze wire fastened to screws at the top and bottom in the head and foot blocks.

Roughing-Filter Washing Devices.—Provision for washing each roughing filter was made through an 8-in. connection with a 12-in. pipe laid especially to bring filtered water to the filter plant from the force main leading from the city pumping station to the service reservoir, with a cross-connection also to the 16-in. return main, so that

water may be drawn from either one or the other of these two pipes as desired. The 12-in. pipe divides, on reaching the filter plant, an 8-in. branch, Plate XLVII, entering the roughing-filter plant at the bottom of the channel which receives the rough-filtered water, with connections with the underdrain of each filter back of the effluent control valve, each of these connections being controlled by its own valve operated by a hand-wheel from the operating floor above. In addition, one other 8-in. branch is taken off from the 8-in. wash-water main and terminates in the sewer chamber of the central filter for use in flushing the sewer, if necessary.

In addition to the wash-water, provision is made for scouring the beds of the roughing filters with air admitted to the underdrains through 2-in. galvanized-iron pipes leading from the air receiver and provided with control valves and pressures gauges. The air pipes connect with the wash-water pipes in the bottom of the filtered-water effluent chamber of each filter between the wash-water control valve and the effluent underdrain.

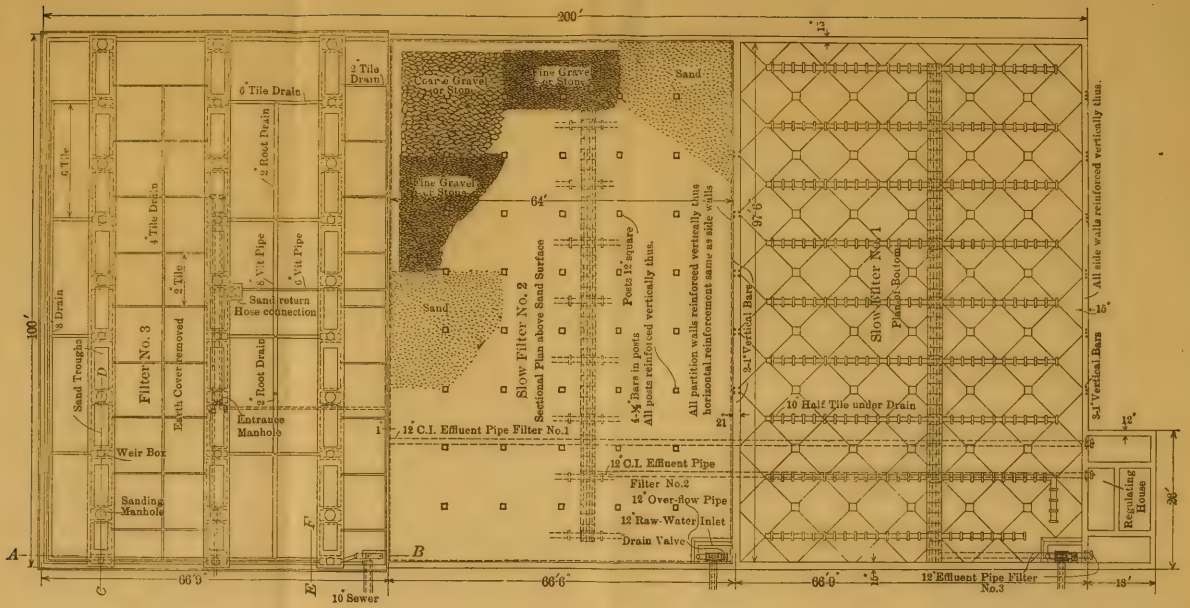
Sewer Connection.—Under the regulating chambers of the roughing filters there is an 18-in. vitrified pipe sewer, Plates XLVII and L, laid in concrete and extending to the river, a distance of approximately 1 000 ft. Into this sewer all parts of the plant can ultimately be drained.

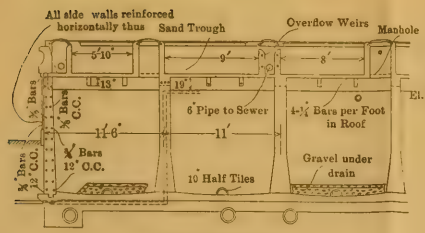
The floor of the operating room of the roughing filters forms the roof over the various regulating chambers and the channel for the rough-filtered water; and the manholes providing for entrance into the various chambers are covered with light, 20-in., circular, cast-iron covers with ring-frames built into the concrete, manhole steps being provided in the walls.

General Water-Supply Pipes.—The water supply for mixing the coagulant solution, for supplying the steam-heating plant, the toilets, and the sand-washer plant of the slow filters, is taken from the 8-in. wash-water main in the effluent channel of the roughing filters by a vertical 3-in. galvanized-iron pipe rising above the ceiling of the operating room and extending lengthwise of the roughing filters to the sand-washer room in the second story of the building over the regulating chambers of the slow filters. The branches for the different fixtures are taken from this main line where required.

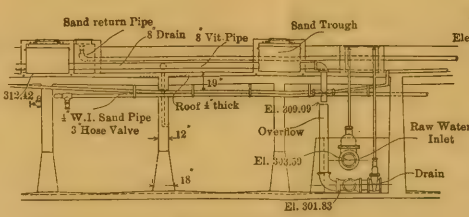
Operating Platform for Chemical Tanks.—From the floor of the operating room of the roughing filters, reinforced concrete stairways,

SLOW FILTERS

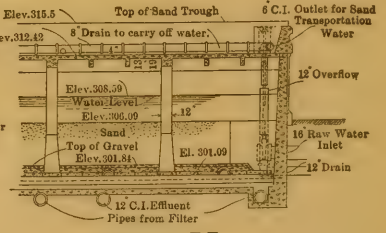




SECTION ON *C-D*



SECTION ON *A-B*



SECTION ON *E-F*

Plate L, lead up on either hand to the elevated platform in front of the coagulant-mixing tanks, and the lime-water mixing tanks. These are provided with iron-pipe railings, painted black and varnished.

Centrifugal Pump.—The centrifugal pump, Fig. 1, Plate LI, which supplies the water through the measuring box to the lime-mixing tanks, has a 2-in. discharge pipe, and a 2-in. suction pipe, extending down into the channel containing the rough-filtered water, having on its lower end an elbow, short nipple, and gate-valve with extension stem reaching up above the floor of the operating room and there provided with a hand-wheel. Just below the floor level of the operating room a 2-in. horizontal branch extends through one of the roughing filters to the slow-filter operating room where a hose-valve for 2-in. suction hose is provided in order to permit of using the centrifugal pump to empty the regulating wells of the slow filters below the elevation to which they could be drained naturally.

Electric Motors.—Power to operate the machinery in the plant is derived from the electric current supplied by the York Haven Power Company, and is of the alternating-current type delivered at a voltage of 220 at the switch-board. The motors, Plate XLIX, of which there are two, manufactured by the Westinghouse Electric and Manufacturing Company, one of 5 h.p. and the other of 15 h.p, are of the alternating-current type, wound for 3-phase, 60-cycle, 200-volt terminal voltage, capable of standing an overload of 25% for 2 hours without injury, and provided with oil-immersion starting boxes. A marbleized-slate switch-board, mounted with volt meter, ampere meter, watt meters, circuit breakers, ground detectors, pilot lights, clock, and main and circuit switches for the different services required, stands in the machinery room.

Lighting.—All the filters and different parts of the plant are provided with electric light fixtures wherever light may be required, and all the rooms of the building containing the machinery and the regulating chambers of the different filters are also provided with gas jets.

Air Compressor.—The air compressor for supplying compressed air for scouring the filtering materials in the roughing filters, and for agitating the coagulant solution, stands between the two motors, and can be operated from either one. It is an Ingersoll-Rand compressor, known as the Imperial Type 11, and consists of two cylinders standing vertically, with single-acting plungers driven by crank shafts on opposite sides of a heavy fly-wheel which also serves as the belt pulley.

There is provided on the outlet from the compressor an unloading device, so-called, which, when the pressure reaches the maximum for which the machine is intended, allows the excess pressure to be relieved automatically.

The air compressor has a capacity of 39 cu. ft. of free air per min. at a speed of 200 rev. per min., and is capable of delivering the air at that rate under a pressure of 100 lb. per sq. in. The air cylinders are water-jacketed, with hooded ends, the cooling water being discharged into the trough of the filter below; the air valves are of the poppet type, and work vertically.

Air Receiver.—A 2-in. galvanized-iron discharge pipe, with a brass check-valve near the compressor, leads from the compressor to the air receiver, which is a vertical cylindrical tank, with domed ends, 5 ft. in diameter, and 12 ft. high. The shell is of flange-steel, is furnished with a manhole and a cast-iron base, and is proportioned to stand a working air pressure of 110 lb. per sq. in. and remain air-tight under that pressure. The receiver is provided with connections for the 2-in. galvanized-iron discharge pipe from the compressor and the 2-in. galvanized-iron air pipe leading from the receiver to the air wash-pipes of the roughing filters, and has also a 1¼-in drip at the bottom, provided with a double valve.

The receiver is provided with a safety valve set at 105 lb. per sq. in., a gauge to indicate the pressure in the receiver, and a 1¼-in. Foster reducing valve capable of discharging the entire contents of the receiver in 2 min. at a uniform pressure on the filter side of the valve of about 5 lb. per sq. in., the pressure of the receiver falling, at the same time, from 100 to about 10 lb. per sq. in.

The two motors, the air compressor, and the air receiver stand on foundations built monolithic with the roof of the west roughing filter, and no vibration is apparent when these machines are operating.

Countershaft.—The countershaft to which the motors and air compressor are belted is provided with a jaw coupler between the belt pulleys from the two motors, and is extended through the south wall of the machinery room into the regulating room of the roughing filters, Plates XLVIII and L, where additional countershafts are installed in order to reduce the speed as required for the operation of the grit elevator, and to increase the speed for the operation of the centrifugal pump supplying the water for the lime-water solution.

FIG. 1.—TOPS OF SLOW FILTERS DURING PLACING OF EARTH AND TOP-SOIL COVERING.

FIG. 2.—INTERIOR OF SLOW FILTER, SHOWING PLACING OF UNDERDRAINS AND GRAVEL LAYERS.

Superstructure.—The building covering the operating room of the roughing filters, the coagulant storage room, the machinery room, and the regulating room of the slow filters, Fig. 2, Plate LIV, is L-shaped, one leg of which is 44 and the other 65 ft. long, measuring on the long sides, the two wings being, respectively, 21 and 26 ft. wide. The outer faces of the side walls are of red brick, white brick being used on the inside faces. The building is surmounted by a slate roof on a timber framing, with copper ridge, finials, gutters, and down-spouts, the down-spouts discharging into the troughs of the roughing filters, or into the deposit chamber, as necessary.

Sampling Devices.—In front of each roughing filter, and by the side of the entrance chamber for raw water, a small concrete bracket, Fig. 1, Plate LI, and Fig. 1, Plate LV, containing a copper funnel in the center, is fastened to the wall, and carries a nickel-plated goose-neck on the discharge pipe of a small rotary hand-pump, the mechanism of which is submerged in the water immediately below the shelf. The pumps are operated by extension shafts carrying nickel-plated hand-wheels. These pumps are used for obtaining samples of the raw water as it enters the plant and of the effluent from each roughing filter.

Slow Filters.

The rough-filtered water is conducted to the slow sand filters through a line of 16-in. wood-stave pipe with 12-in. connections to each slow filter, controlled by sluice-gates with stems extending to the tops of the filters under a manhole.

Area.—The slow filters, Plate LII, and Fig. 2, Plate LIV, of which there are three, with a total net area of filtering surface of 0.4340 acre, lie side by side immediately north of the roughing filters. They are each 97 ft. 6 in. long, the two end filters being 64 ft. 7½ in., and the central filter, 64 ft. 9 in. wide, all dimensions being measured inside at the bottom, and all interior wall surfaces battering out toward the top 6 in. in their height. The thickness of the outside walls of the filters is 15 in. at the bottom, and 9 in. at the top; the thickness of the division walls is 21 in. at the bottom and 9 in. at the top. The walls are reinforced horizontally and vertically to take the necessary strains, and are built in sections ending against headers, with copper strips in all vertical joints to cut off any possible leakage at such points.

The floors of the filters are laid in blocks, and form inverted groined arches, with the columns supporting the roof standing on the high points.

Roof, Side Walls, and Floors.—The roof of each filter is supported on forty reinforced concrete posts, 12 in. square from the roof to a point $4\frac{1}{4}$ ft. from the bottom, battering out to a width of 18 in. on each side at the floor level. Longitudinally of each filter, reinforced concrete beams, 19 in. deep, run across the tops of the columns in alternate rows, while transversely of the filters, beams, 12 in. deep, run across the tops of all the columns in each row; on the intermediate lengthwise rows of columns the functions of beams for carrying the roof are performed by the deep sides of the troughs intended for the storage of sand removed from the filters during the periodical scrapings, the roof of the filter and the intersecting roof beams being tied to, and suspended from, the side walls of these troughs by vertical steel rods.

The reinforcement of the side walls of the filters consists of three vertical 1-in. bars, opposite each panel point of about 11 ft., running from top to bottom of the wall, the horizontal reinforcement consisting of $\frac{3}{4}$-in. bars, 12 in. from center to center, for the lower 8 ft. in depth, and $\frac{5}{8}$-in. bars, 12 in. from center to center, for the remainder of the height, bars being used in both faces of each wall. The side walls rest in grooves formed along the outer edge of the floor slabs when the latter were laid. In laying the floor slabs, no particular precaution was taken to prevent leakage, further than to form grooves in each side of each slab as it was laid, allowing plenty of time for the concrete to harden and contract, before laying the next slab against it, and using a grillage of $\frac{1}{2}$-in. bars, 4 ft. long, in the concrete of the floor under the bottom of each post supporting the roof. The grooves in the edges of the floor blocks were made V-shaped so that the settlement of any particular block would tend to wedge the joints tight.

Sand Troughs.—The sand troughs, Fig. 2, Plate LI, and Fig. 1, Plate LIII, on top of the filters, of which there are three to each filter, are divided by cross partitions into boxes from 8 to 9 ft. long, separated at one end by weirs and at the other by manholes opening into the filter. The boxes are arranged so that the weirs of two adjacent boxes stand 12 in. apart, forming a channel crosswise of the sand box, from one end of which leads a 6-in. drain-tile connecting with a

FIG. 1.—INTERIOR OF SLOW FILTER, SHOWING SAND IN PLACE.

FIG. 2.—TOPS OF SLOW FILTERS, SHOWING SAND TROUGHS.

system of drains running over the tops of the filters and discharging to the sewer through the overflow of each slow filter. The sand boxes have covers of ¼-in. checkered steel plates stiffened by angle irons riveted to the underside thereof, the cover-plates being in two halves for convenience in handling. Manhole heads and covers, 18 in. in diameter, are provided for the compartments containing the weirs, and 24-in. heads and covers for the manhole opening into the filter, except for the main entrance, for which there is a rectangular checkered steel cover, 2 by 4 ft. The sand boxes have 2-in. drain-tile connections at the bottom of each, and lead into the 4-in. tile drains between two adjacent rows of sand boxes; these connections allow the water to drain out of the sand when the box is filled from the sand-washing plant.

Frost Proofing.—Around the periphery of the plant a curb, 2.5 ft. high, of ornamental design was built on top of the roofs of the filters, and the entire roof, after the pipe drains were laid, was covered with earth, Fig. 1, Plate LIII, to a depth of 2.5 ft., the upper 9 in. being of top soil. The whole area was then raked over and sowed with grass seed.

Exterior Finish.—The visible exterior faces of the slow filter walls were divided into panels by pilasters moulded monolithic with the walls, the effect being carried around the roughing filters as a water-table from which to start the brickwork of the building containing the regulating rooms, storage rooms, etc.

Supply and Drain Valves.—In the southwest corner of each of the three slow filters is a compartment, Plate LII, the top edge of which is level with the surface of the sand. The rough-filtered water is delivered to the filters through these compartments. Drains for emptying the water from the surface of the filters, and overflow pipes connecting with a 12-in. drain, lead to the sewer. The drains from the sand troughs on the roof discharge vertically downward into the overflow pipe.

Slow-Filter Underdrains.—The underdrainage system of the slow filters consists, Plate LII, and Fig. 2, Plate LIV, of lines of 10-in. half-tiles lying at right angles to the length of the filter in the center of each panel between the rows of posts, and connecting with a main underdrain running lengthwise of the filter in one of the panels adjacent to the center line; a 12-in. cast-iron pipe leads from

this central drain to the regulating house. The effluent control valves for all three filters are placed in a special house forming a part of the structure standing on the roughing filters. The half-tile drains were extended to a point about 4.5 ft. from the walls, all around, the ends being closed, and where they cross the central underdrain they were covered with concrete slabs forming part of the tight cover of the main underdrain. The vitrified hub and spigot tile drains were laid with open joints, and were surrounded with layers of broken stone, the first or bottom layer, about 6 in. thick, being composed of fragments of crushed sandstone the largest pieces of which were not more than 3 in. in diameter and the smallest not less than $\frac{3}{4}$ in. in diameter; the second layer, 3 in. thick, was formed of sandstone particles from $\frac{1}{4}$ to $\frac{3}{4}$ in. in diameter, from which the dust had been removed and in which there were few particles larger than $\frac{1}{2}$ in. in diameter; and the top layer, also 3 in. thick, was prepared from sandstone screenings, from which the dust had been removed and in which there were no particles larger than $\frac{1}{2}$ in. in diameter and very few larger than $\frac{1}{4}$ in. in diameter. The material was secured from a quarry at Marysville, on the Susquehanna, some miles above Harrisburg, and was in three sizes, $\frac{3}{4}$ in. to 3 in., $\frac{1}{4}$ to $\frac{3}{4}$ in., and screenings, the latter containing a considerable quantity of fine dust as well as particles up to $\frac{3}{4}$ in. in diameter. By proper sorting the only material requiring a second handling was the screenings, which, on sifting through $\frac{1}{4}$-in. and $\frac{3}{4}$-in. sieves, provided for all the fine material and for nearly one-half the $\frac{1}{4}$ to $\frac{3}{4}$-in. gravel required; but the weight of the wasted dust from the screenings exceeded, by approximately 25%, the weight of the useful materials secured. As received at the works, the materials were quite dirty, and a simple plan for washing out the stone dust was devised by the contractor's foreman. This consisted of a couple of oil barrels mounted tandem on horizontal axes slightly above the centers of the barrels, on a framework high enough to allow a dump-cart to stand thereunder. The two barrels were provided with a simple lever arrangement for turning them upside down. After being filled with water from a hydrant, they were suddenly upset on a cart containing, perhaps, $\frac{1}{2}$ cu. yd. of broken stone. This quick flush effectively washed the mud from the whole depth of stones in the cart, the surplus water draining out under the tailboard.

The gravel underdrains were not carried out to the side walls of the

FIG. 1.—SLOW-FILTER OPERATING ROOM.

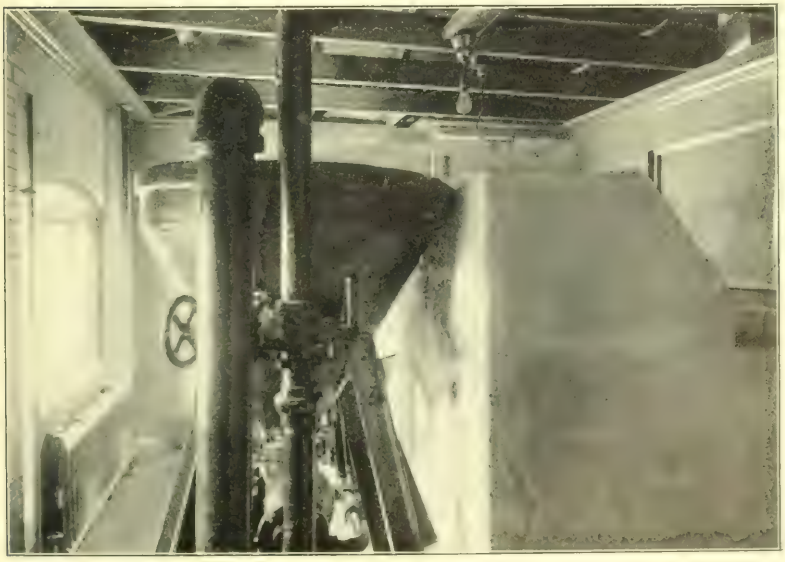


FIG. 2. SAND WASHER.

filter, but were stopped about 2 ft. therefrom and banked up to the proper height, the finer layers of gravel covering the coarser layers on the slopes, and the top surface being brought to a uniform level.

Filter Sand.—The filter sand was secured from the Susquehanna River, and required no further preparation than screening, and, at times, the mixing of sands obtained from different localities.

The specifications for the sand were as follows:

Not more than 10% should be smaller than 0.29 mm. in diameter.
Not more than 5% should be smaller than 0.24 mm. in diameter.
At least 90% should be finer than 0.80 mm. in diameter.

Practically all the particles should be finer than 1 mm. in diameter, and the uniformity coefficient should be not greater than 1.6 to 1.8.

These specifications were written for the Susquehanna River sand with which the writer had had much experience; a modification of these requirements might be desirable for sands from other localities.

As to chemical composition, the Susquehanna River sands were satisfactory, containing less than 2% of lime and magnesia, and less than 5% of an aggregate amount of iron and aluminum in forms likely to disintegrate under the action of water or chemical changes produced by matters the water might contain when passing through the sand. The analyses of the samples of sand taken after the filters were ready to be put in operation are given in Table 1.

Filter No. 3 and practically all of Filter No. 2 were filled with sand obtained at Northumberland, from the West Branch of the Susquehanna, and required no further preparation than the careful selection of the barge loads as the sand was dredged, accepting those which would fill the requirements satisfactorily, as indicated by field tests conducted as the dredging proceeded, and rejecting those in which the sand was too fine or too dirty. The only difficulty experienced with this sand was the likelihood of its containing noticeable quantities of saw-dust, shreds of bark, and friable particles, but casting it by hand over screens having ¼-in. meshes removed the objectionable matters to a degree which rendered the sand satisfactory for filtration purposes.

Owing to the frequent changes in the conditions of flow in the Susquehanna, the character of the sand deposited is subject to variation, and, before a sufficient quantity had been secured, the Northumberland deposits had become too fine for use and it was necessary

to seek a coarser sand to mix with it in order to increase its effective size. An inspection of several deposits farther up the river finally disclosed one at Nesbit, from which much of the finer material had been washed, and by combining the mechanical analyses of this and the Northumberland sands, it was found that a satisfactory composite sand could be made by mixing the two in proper proportions.

TABLE 1.—MECHANICAL ANALYSES OF SAND IN SLOW FILTERS OF STEELTON FILTER PLANT.

Note.—In this table the figures in Column 1 give the depth from which the sample was taken, those in Column 2 refer to samples collected on the northeast corner of the filter, those in Column 3 to samples of the southeast corner, those in Column 4 to samples collected in the center of the filter, those in Column 5 to samples collected in the northwest corner of the filter, and those in Column 6 to samples collected in the southwest corner of the filter.

(1)	(2)		(3)		(4)		(5)		(6)	
	NORTHEAST CORNER.		SOUTHEAST CORNER.		CENTER.		NORTHWEST CORNER.		SOUTHWEST CORNER.	
Depth from which sample was taken.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.
FILTER NO. 1.										
Top 0.5 ft.	0.32	1.7	0.32	1.5	0.32	1.7	0.31	1.5	0.31	2.0
0.5 to 1.5 ft.	0.30	1.8	0.30	1.6	0.31	1.7	0.31	1.6	0.31	1.9
1.5 to 3.0 ft.	0.29	1.9	0.32	1.5	0.32	1.5	0.30	1.6	0.32	2.0
3.0 to 4.0 ft.	0.31	1.5	0.33	1.3	0.29	1.7	0.31	1.6	0.32	1.8
FILTER NO. 2.										
Top 0.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.31	1.4	0.30	1.4
0.5 to 1.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.30	1.5	0.30	1.4
1.5 to 3.0 ft.	0.30	1.4	0.31	1.4	0.30	1.5	0.31	1.4	0.30	1.4
3.0 to 4.0 ft.	0.31	1.4	0.40	1.3	0.31	1.4	0.30	1.4	0.29	1.5
FILTER NO. 3.										
Top 0.5 ft.	0.31	1.6	0.30	1.8	0.31	1.8	0.31	1.7		
0.5 to 1.5 ft.	0.32	1.6	0.30	1.8	0.33	1.7	0.29	1.8	Samples	
1.5 to 3.0 ft.	0.32	1.6	0.29	1.7	0.36	1.5	0.30	1.8	not taken.	
3.0 to 4.0 ft.	0.31	1.7	0.32	1.7	0.36	1.5	0.28	1.8		

As an example, Table 2 exhibits the analyses of the Northumberland and Nesbit sands on July 2d, 1908, at which time the mixing of the two sands in equal proportions produced the desired composite.

TABLE 2.

Sieve: Meshes, per inch.	Size of separation of sieves, in millimeters.	Nesbit sand. Percentage passing sieve.	Northumberland sand. Percentage passing sieve.	Composite sand. Percentage passing sieve.
20	1.02	100	100	100
30	0.57	64	65	65
50	0.38	8	23	13
60	0.29	2	14	7
80	0.21	0.3	0.6	0.5
Effective size		0.4 mm.	0.27 mm.	0.32 mm.
Uniformity coefficient		1.4	2.00	1.6

When Filter No. 1 was being filled, the Northumberland sand had an effective size of only about 0.27 or 0.28 mm. The two sands were mixed by dumping cart loads of each kind upon separate platforms on each side of a manhole, and casting simultaneously the correct relative numbers of shovelsful from each pile upon a screen standing over the manhole. In passing through the screen and falling to the filter below, the two sands became thoroughly mixed, as will be seen from an inspection of the analyses in Table 1.

The sand was delivered to the filters in one-horse, two-wheeled carts. It was dumped through the manholes, and spread in three layers, care being taken not to compact it unevenly. The top surface, Fig. 1, Plate LIV, was leveled off with straight-edges to a practically uniform elevation, the final operation being to rake the entire area over lightly with an iron rake. A typical mechanical analysis of the Northumberland sand, when running at its best, is given in Table 3.

The Northumberland sand, during the process of dredging from the river bottom, was quite thoroughly washed, and required no further treatment except screening through ¾-in. mesh screens to remove mussel shells, shreds of bark, etc., before being placed in the filters.

The total quantity of sand shipped to Steelton for the slow filters was 3 989 tons, in 107 car loads, and made, in the filters, 2 953 cu. yd.

In order to help out the contractor in the matter of securing this

sand, the writer kept an inspector at the sand dredges, whenever sand was being secured, during the entire period covering November and December, 1907, and January, April, May, June, and July, 1908; actual dredging operations, however, were limited to 92 days during this time.

TABLE 3.—MECHANICAL ANALYSIS OF SAMPLES OF NORTHUMBERLAND SAND, OCTOBER 31ST, 1907.

Size of sieve. Meshes, per linear inch.	Effective size of separation, in millimeters.	Weight of sand passing sieve, in pounds.	Percentage passing.
4	3.4	2,295	100
10	2.15	2,289	99.7
20	1.02	2,274	99
30	0.57	2,183	95
40	0.45	1,774	76
50	0.38	1,067	46
60	0.29	0,170	7.4
80	0.21	0,025	1.1
100	0.17	0,012	0.6
150	0.12	0,005	0.0

Note.—Effective size, 0.31 mm.; uniformity coefficient, 1.37: finer than 0.24 mm., 2.5%; finer than 1 mm., 99%; finer than 1.5 mm., 99.5 per cent.

The total cost of inspecting the sand, including the salary of the inspector, his board, and traveling expenses, was 9.5 cents per ton, or 13.0 cents per cu. yd. As the sand was running, a satisfactory indication of its character was obtained by a mechanical analysis of one sample to about each 30 cu. yd. dredged. The sand was sampled on the barge before unloading, each sample being a composite of samples collected at about four places from freshly excavated vertical faces in the sand as piled on the barge, and collected so as to be representative of the material for the full depth of the pile. These samples were then thoroughly mixed, the sample for analysis being taken therefrom, dried, and sifted through carefully rated screens.

Sand obtained in the river deposits in the neighborhood of Harrisburg could not have been used without expensive preparatory treatment to remove the fine material, as its effective size usually runs from about 0.23 to 0.24 mm. To remove some of the fine material, washing is necessary, the total waste ranging from about 10% to more than 50%, depending largely on the quantity of coal dust in the sand. The sand from Northumberland, being obtained from the West Branch above the junction of the North Branch, is free from coal, and is also coarser than that obtained farther down the river.

Effluent Control of Slow Filters.—The effluent pipes pass beneath the floors of the slow filters, and each terminates in its own regulating chamber with a sluice-gate controlled from a valve-stand and hand-wheel on the floor of the operating room. The rate of filtration of each filter is controlled by causing the filtered water to flow through a submerged orifice, Fig. 4, 8 in. in diameter in the end of an outlet pipe having a swivel-joint at the lower end, Plates XLVII and XLVIII, and discharging through a gate-valve into a filtered-water well common to the three filters. The free or orifice end of the swivel-

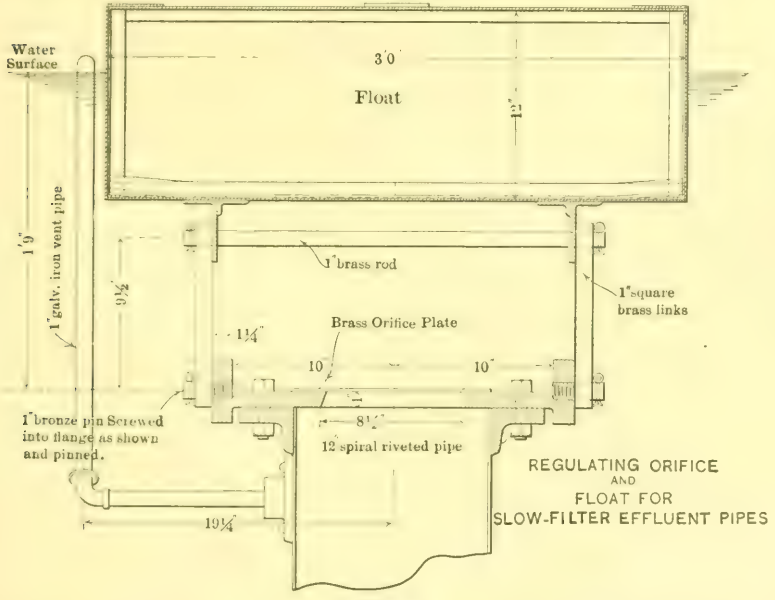


FIG. 4.

jointed pipe is suspended by brass links from a float resting on the surface of the filtered water in the effluent chamber of the filter; the center of the 8-in. orifice is adjusted to stand 1 ft. 9 in. below the surface of the water in the effluent well, at which depth, with a free discharge through the orifice into the atmosphere, the filter will deliver 1 500 000 gal. per day to the outer well. If the pumps in the Borough pumping station do not take the water away as fast as the three filters can furnish it, the water rises in the outer well, submerges the orifices, and cuts down the head by an amount just sufficient to keep up the supply, the plant being capable, therefore, of yielding the water to the

pumping station automatically at any rate necessary, up to the maximum for which the plant is designed, namely, 1 500 000 gal. per day per filter. In order to prevent any draft-tube effect in the effluent pipe, between the effluent well and the outer filtered-water well, a 2-in. galvanized-iron pipe is connected with the swivel-jointed pipe at a point below the orifice, the free end of the 2-in. pipe being adjusted so as to be always above the water level in the effluent well, whatever position the float may take; this assures a discharge into the atmosphere whenever the difference in level of the water between the effluent well and the filtered-water well of any filter exceeds 1 ft. 9 in.

Gauge Boards.—The rate of filtration and loss of head are indicated on scale boards, Fig. 1, Plate LV, carried by bronze wires attached to floats in the different compartments of the regulating house, as described for the roughing filters. A sampling pump, similar to those for the roughing filters and the raw water, is provided for the effluent well of each slow filter.

The filtered water leaves the central filtered-water well through a 24-in. wood-stave pipe leading to the new pump-well at the pumping station, a sluice-gate being provided at each end of this pipe line.

Sand Washer.—The system of handling the sand when the slow filters require cleaning is not new in principle, but the details differ materially from those of other plants. Portable ejectors, supplied with water at about 100-lb. pressure from a line of 4-in. wrought-iron pipe, Plate LII and Fig. 2, Plate LIII, suspended from the ceiling of each slow filter, through 3-in. bronze hose-gates and 3-in. hose, lift the dirty sand and transport it to the sand-washer plant. The dirty sand, having been shoveled into piles in the filter, is cast into the hopper of the portable ejector, is picked up by the jet of water passing through the ejector, and delivered through a line of 4-in. wrought-iron pipe to the sand-washing hoppers, Plate XLVIII, and Fig. 2, Plate LV, in the second story of the slow-filter regulating house. After passing through two hoppers the washed sand is forced by a jet in the bottom of the second hopper through a line of 4-in. wrought-iron pipe, resting on the roof of the three filters, to hose-valves from which lines of hose may be laid to discharge the returning clean sand into the compartments of the sand troughs running lengthwise of the tops of the filters. The operation is simple, and requires no special description or comment.

Heating.—The building covering the operating rooms of the rough-

ing and slow filters, as well as the machinery room and the coagulant-storage and sand-washer rooms, is heated by steam generated in a vertical tubular boiler standing in the second story of the slow-filter regulating house. The returns from the radiators are piped back to a trap in one corner of the slow-filter operating room, and discharge into the sewer. The provisions for heating required boiler capacity with sufficient heating surface and grate area to furnish steam to raise the temperature of 20 000 lb. of coagulant solution from 32° to 70° Fahr., in 1 hour. The boiler is designed to carry 100 lb. of steam, and is provided with a reducing valve, on the steam-heating connection, set at 5 lb. The coils and radiators are proportioned to maintain the machinery and operating rooms at a temperature of 70° and the coagulant-storage room and the sand-washer room at a temperature of 58° when the outdoor temperature is at zero. The toilet-room is provided with a wash-basin with cold-water connection and a steam connection for securing hot water, and a low-down tank closet.

Construction.

No special difficulties were encountered during construction, the bearing value of the foundations proving satisfactory for the relatively light loads they are required to carry. There was comparatively little ground-water in any of the work, although the permanent ground-water level was barely below the floor of the excavation for the slow filters; in fact, a modification of the design of the main underdrains of two of the filters was made necessary in order to keep the work out of the water. A considerable portion of the site of the slow filters was, in early days, a low swale which had been used subsequently as a dumping ground for slag, cinders, and ashes from the steel works, and it was owing to the presence of these materials, which had had such a long time to become thoroughly settled, that satisfactory foundations were secured without extra work.

Leakage.—Before the sand was placed in the slow filters they were filled with water from the street mains up to the level required for operation, the water from the street being then shut off and the drop in the surface level being observed. It was not expected that the filters would be perfectly tight, owing to the construction of the bottoms in blocks about 11 ft. square, with no special provision for stopping leakage between the blocks. The leakage from each of the

three filters was about the same, and aggregated about 0.66 of 1% of the nominal daily capacity of the plant. No tests to determine the amount of leakage have been made since the filters have been in operation, but it is believed that there has not been an increase.

Cost of the Plant.—The plans were completed in July, 1907, and, in response to advertisements, three bids were opened on August 7th, 1907, aggregating $71 522.85, $80 482.48, and $89 592.59, respectively. The contract, which was awarded August 14th, 1907, to the Bunting Construction Company, of Flushing, N. Y., stipulated that the work should be completed within 6 months from the date of the contract, or by February 14th, 1908. Various delays resulted, however, and it was not until the middle of September that the plant could be put in operation, the final estimate to the contractor being dated September 24th, 1908, and amounting to $70 730.87. This, however, did not represent the full cost of the work, as the installation of the centrifugal pumps, the changing of the suction pipes at the pumping station, and some repair work on the checkered steel covers on the slow filters, were done by the Water Board outside of the contract. In addition to the above items, there were the cost of the site and of engineering and incidental expenses, which were high, by reason of the fact that the time occupied by the contractor in the construction of the plant was more than twice that in which the work should have been completed under favorable conditions.

The following are the principal items making up the cost of the plant:

Excavation, grading, roadways, sodding, etc..	$3 936.88
Sewers and drains	2 314.67
Concrete, all classes	18 708.32
Reinforcing steel	3 475.80
Manhole heads and covers, indoors and outdoors	1 590.50
Cast-iron pipes and specials	3 956.27
Wood-stave pipes between filter plant and pumping station	9 034.21
Filter regulating devices, gauges, coagulant and lime-water apparatus	2 650.00
Superstructure over regulating houses, etc., including steam heating, lighting, and plumbing	7 900.00

Machinery, plant, motors, compressor, shafting, belting, air receiver, sand-washing plant, etc.	3 750.00
Gate and sluice valves	1 485.00
Filtering materials, including underdrains	10 811.34
Extra work	1 117.88
	$70 730.87

Of the above amount, $57 000.00 is the cost of the purification works, the remainder, $13 700.00, being the cost of the pipes to the pumping station, the sewer to the river and other details growing out of local conditions.

Operation.

The plant was put in service on September 12th, 1908, but was shut down later in the day, and was started again on September 16th, since which date it has furnished filtered water continuously.

The regular force required for operation consists of a general superintendent and two filter attendants, the latter working on 12-hour shifts. When the slow filters require scraping an additional force is taken on temporarily, the men working under the direction of the general superintendent.

The permit for the construction of the plant, issued by the Pennsylvania State Department of Health, provided that the filters should be operated for a considerable period under the direction of its designer, and in compliance with this condition the writer instructed the superintendent and attendants in their duties, provided specific instructions regarding the manipulation of the roughing and slow filters, and the handling of the coagulant, and has advised with reference to special features of the operation whenever necessary.

From September 16th, 1908, until January 9th, 1909, no coagulant was used in connection with the operation of the roughing filters, the raw water being sufficiently free from turbidity to yield a satisfactory water without coagulant.

Quantity of Coagulant Required.— The general instructions given to the superintendent called for the application of a coagulant to the raw water whenever its turbidity exceeded 50 parts per million, and the continuance of its use for much lower turbidities whenever the

last bacterial analyses showed 5 000 or more bacteria per cu. cm. in the raw water, or in case the effluent from the slow filters showed objectionable color. It is the intention, in dosing with coagulant, to use a sufficient quantity to produce an effluent from the roughing filters having a turbidity of not more than 10% of that of the applied water, and not, in any case, more than 25 parts per million, regardless of the turbidity of the raw water. It has also been found desirable, particularly during the winter and the cold spring weather, to manipulate the dose of coagulant so as to produce an effluent from the rough-

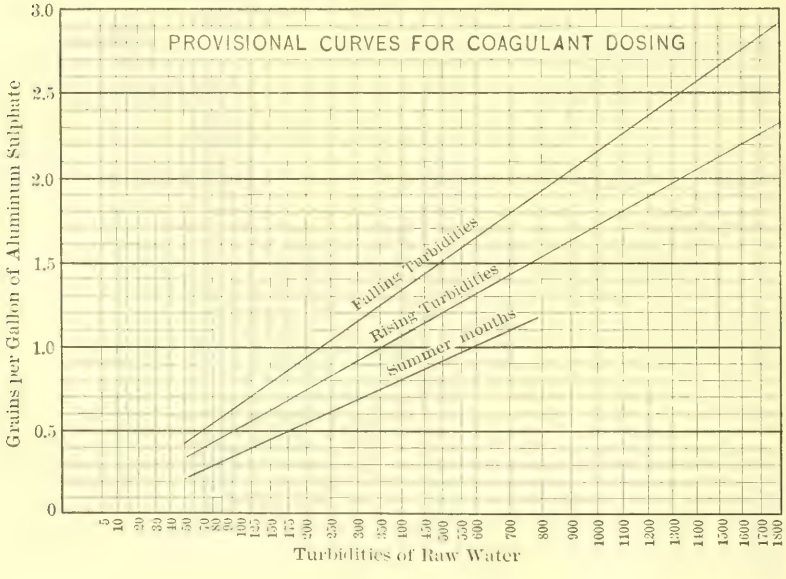

PROVISIONAL CURVES FOR COAGULANT DOSING
Grains per Gallon of Aluminum Sulphate
3.0
2.5
2.0
1.5
1.0
0.5
0
Falling Turbidities
Rising Turbidities
Summer months
5 10 20 30 40 50 70 80 90 100 125 150 175 200 250 300 350 400 450 500 550 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800
Turbidities of Raw Water

FIG. 5.

ing filters having a turbidity of practically zero, as by this means the runs of the slow filters are very greatly lengthened and the cost of operation is not increased by the use of the comparatively small additional amount of coagulant necessary to secure this result.

The quantity of coagulant to be used is determined by the superintendent from a diagram, Fig. 5, prepared by the writer from data secured originally during the operation of the Harrisburg Testing Station, in 1902 and 1903, but modified somewhat to take into account the difference in the character of the filtering materials, and the slightly different character of the raw water at the Steelton intake; the

diagram may require further modification as additional knowledge is gained during the operation of the plant. The dose of coagulant indicated by the lowest curve is for application during the summer when the turbidities do not rise very high, or very suddenly. The dosing indicated by the middle curve is required when the turbidity is rising rapidly, during the first parts of the floods in the fall, winter and spring; the dosing indicated by the upper curve is required when the turbidity is falling, after the passage of a flood. It will be noticed that for equal turbidities more coagulant is required when the turbidities are falling than when they are rising, on account of the finer character of the turbidity and the persistence of high numbers of bacteria in the Susquehanna after the turbidities have begun to fall. The superintendent must use judgment in determining the proper dose of coagulant. The character of the turbidity of the river water varies so greatly and changes so quickly that a dose satisfactory with a given clay turbidity, say, of 500 parts per million, might prove altogether too much for an equal turbidity from the North Branch. At times, also, particularly after a protracted season of low water, a turbidity caused largely by the fine shreds of vegetation torn and scoured loose from the rocky river bottom is particularly difficult to handle by reason of its high clogging value when it becomes matted upon the filter surface. In order to keep the plant in efficient operation, therefore, the superintendent must watch the effect of his dosing and reduce the quantities if he finds the loss of head increasing on the roughing filters too rapidly, or increase the quantity if the conditions require it.

When to Wash the Roughing Filters.— If, when coagulant is being applied to the raw water in the proper amount, the effluent of a roughing filter shows objectionable turbidity, indicating the passage of coagulated material through the filter bed, the filter is immediately washed; otherwise, it is left in operation until the maximum allowed loss of head, about 2.5 ft., is reached.

This method of operation is based on the theory, which seems to be supported by considerable evidence gained both at this plant and at the Harrisburg Testing Station, that these coarse-grained filters act very much as settling basins with bottom areas of great extent. The mud carried into the filter by the entering water is deposited on the granules of the filter bed through practically its entire depth, and one

of the requirements in controlling their operation is to prevent the washing of this sediment through the filters by pushing their operation too rapidly, or by using too great losses of head, or by suddenly increasing the rates of filtration. To prevent most of these evils the mechanisms at the plant limit the loss of head to about 2.5 ft.

Regulation of Coagulant Dosing.—The quantity of solution required to supply a given dose of coagulant depends on the quantity of water being filtered and the percentage strength of the solution.

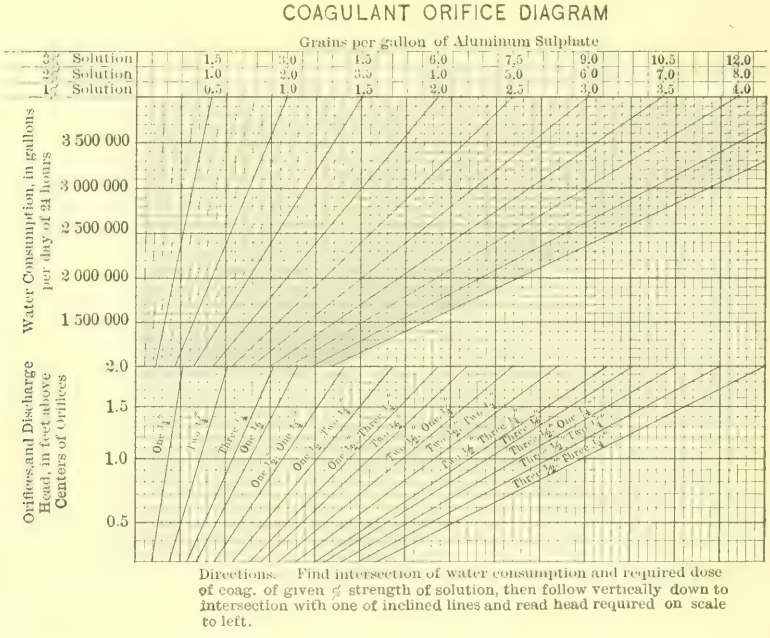

COAGULANT ORIFICE DIAGRAM
Grains per gallon of Aluminum Sulphate
3% Solution 1.5 3.0 4.5 6.0 7.5 9.0 10.5 12.0
2% Solution 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0
1% Solution 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0
Water Consumption, in gallons per day of 24 hours
3 500 000
3 000 000
2 500 000
2 000 000
1 500 000
Orifices and Discharge Head, in feet above Centers of Orifices
2.0
1.5
1.0
0.5
One ¼ Two ¼
Three ¼
One ½ One ¼
One ½ Two ¼
One ½ Three ¼
Two ½ One ¼
Two ½ Two ¼
Two ½ Three ¼
Three ½ One ¼
Three ½ Two ¼
Three ½ Three ¼
Directions. Find intersection of water consumption and required dose of coag. of given % strength of solution, then follow vertically down to intersection with one of inclined lines and read head required on scale to left.

FIG. 6.

Knowing these, the requisite quantity of solution is measured out by continuous discharge through orifices in the glass plate forming the front of the measuring box, the size and number of orifices, as well as the head required to give the necessary dose, in grains per gallon, being ascertained by the superintendent from a diagram, Fig. 6, prepared for his use. The lines indicating the orifices to be used were placed on the diagram in accordance with the measured discharge of each orifice.

Use of Lime-Water.—During floods the alkalinity of the river water

falls sometimes too low to permit of the decomposition of the requisite dose of aluminum sulphate, and provisions are made to add, at such times, sufficient milk of lime to supply the deficiency in alkalinity. When lime must be used, enough is added to neutralize the free CO_2 in the river water, combine with the aluminum sulphate, and leave a residual alkalinity in the water of about 6 parts per million. The free CO_2 and alkalinity are determined by the methods recommended by the Committee on Standard Methods of Water Analysis, of the American Public Health Association. Knowing the dose of aluminum sulphate being used, and having ascertained the alkalinity of the raw water, and the amount of free CO_2 carried, reference to a diagram including all these data will give the quantity of lime required, if any.

Regulation of Lime Dosing.—The quantity of water required to add this dose of lime to the raw water in a saturated solution is regulated at the lime-water measuring box in the manner in which the coagulant dose is regulated, a diagram similar to the coagulant-dosing diagram being supplied for the lime-water. The amount of lime to be slaked daily to produce the required quantity of lime-water, for different dosings and different daily consumptions of water, is determined by the superintendent from another diagram, small quantities being slaked from time to time during the day and discharged as milk of lime through the funnel and pipe leading to the bottom of the lime-mixing tanks. The strength of the lime-water solution is to be tested from time to time by the usual simple chemical test, and it should be kept up to full strength by the attendant.

Another diagram gives the number of pounds of aluminum sulphate required for making solutions of different percentage strength, the capacity of the tanks having been calibrated for the purpose.

Simplicity of the Dosing System.—No calculations being required on the part of the superintendent during the operation of the plant, the rapid changing of the dosing, when required, can be effected with accuracy and without the likelihood of errors from faulty calculations.

The lime-water and the coagulant solution are admitted to the water practically together. The conditions requiring the use of lime being comparatively rare no great pains have been taken to go into refinements in the use of lime that would be desirable under conditions requiring its more continuous use.

Reduction of Alkalinity by Coagulant.—As to the proportion of the coagulant rendered inactive through its absorption by suspended matters in the water, sufficient data have not yet been obtained at the Steelton plant to speak authoritatively, but it is believed, from present indications, that the reduction of alkalinity of the raw water, instead of the theoretical 8.2, will probably be about 6 parts per million, per grain of coagulant used, as observed at the Harrisburg Filter Plant during the past three years and at the Harrisburg Testing Station in 1902 and 1903.

Operation of Roughing Filters.

Operation with Respect to Lengths of Runs.—While the roughing filters must be operated in a manner to secure an effluent sufficiently free from turbidity and bacteria to be handled satisfactorily by the slow filters, their operation must be controlled in a manner to secure sufficiently long periods of operation between washings, otherwise the plant could not be kept going during periods of very turbid water. A little experience is required to enable this to be done, but it presents no practical difficulty with any range of conditions thus far encountered on the Susquehanna at Harrisburg and Steelton. The highest turbidity experienced since the plant has been in operation occurred in the forenoon of January 26th, reaching 1 600 parts per million, remaining at that figure for about 5 hours, and dropping by steps to 500 parts during the next 12 hours. The shortest run during this period was about 2½ hours, which occurred after the turbidity had dropped to about 700 parts. The first turbid water the plant was required to handle occurred about January 8th, at a time when the writer was not able to be present, and, owing to the deterioration of certain of the silica-turbidity standards, the turbidities of the raw water and of the effluents from the roughing filters were incorrectly read, the actual turbidities being much higher than the figures recorded. As a result, the quantity of coagulant used was entirely too small, and a considerable number of bacteria passed through both the roughing filters and the final filters.

Records of Operation.—Since January 23d, 1909, with but very few exceptions, the coagulant has been properly applied, and the satisfactory results obtained in the removal of turbidity are exhibited in Table 4, which gives the hourly records of the operation of Roughing Filter No. 1, through runs Nos. 58 to 81, January 26th to 31st, 1909, inclusive.

TABLE 4.—Hourly Records of Operation of Roughing Filter No. 1 Through Runs Nos. 58 to 81, January 26th to 31st, 1909.

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
58	9	58	Jan. 26	6 A.M.	80	1	98.8	0.42
				7 "	800	1	99.9	0.42
				8 "	800	1	99.9	0.92
59	4	16		9 "	800	1	99.9	1.84
				10 "	1 600	1	99.9	2.22
				11 "	1 600	1	99.9	2.22
				12 M.	1 600	1	99.9	2.22
				1 P.M.	1 600	1	99.9	2.22
60	3	02		2 "	1 600	1	99.9	2.22
				3 "	900	1	99.9	2.00
				4 "	900	1	99.9	2.00
61	2	31		5 "	900	1	99.9	2.00
				6 "	900	1	99.9	2.00
				7 "	800	1	99.9	2.00
62	2	26		8 "	700	1	99.8	1.50
				9 "	700	1	99.8	1.50
63	4	35		10 "	700	1	99.8	1.50
				11 "	700	1	99.8	1.00
				12 "	500	1	99.8	0.75
			Jan. 27	1 A.M.	500	1	99.8	0.75
				2 "	500	2	99.6	0.75
64	2	58		3 "	500	2	99.6	0.75
				4 "	500	2	99.6	0.75
				5 "	500	1	99.8	0.75
65	2	11		6 "	500	1	99.8	0.75
				7 "	600	2	99.7	0.63
				8 "	750	2	99.7	0.63
66	3	18		9 "	800	3	99.6	0.63
				10 "	800	2	99.8	0.63
				11 "	750	2	99.7	0.63
				12 M.	750	2	99.7	0.63
67	3	17		1 P.M.	700	2	99.7	0.63
				2 "	600	3	99.5	0.63
				3 "	550	2	99.6	0.63
68	4	15		4 "	550	2	99.6	0.63
				5 "	550	2	99.6	0.63
				6 "	550	3	99.4	0.63
				7 "	550	3	99.4	0.63
69	3	53		8 "	550	3	99.4	0.63
				9 "	550	3	99.4	0.63
				10 "	550	3	99.4	0.80
				11 "	500	10	98.0	1.95
				12 "	500	2	99.6	1.95
70	3	00	Jan. 28	1 A.M.	500	1	99.8	1.50
				2 "	500	1	99.8	1.50
				3 "	500	1	99.8	1.50
71	4	03		4 "	500	1	99.8	1.50
				5 "	500	1	99.8	1.50
				6 "	500	1	99.8	1.50
				7 "	500	1	99.8	1.30

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
72	4	04	Jan. 28.	8 A. M.	500	1	99.8	1.30
				9 "	500	1	99.8	1.30
				10 "	500	1	99.8	1.30
				11 "	500	1	99.8	1.22
73	4	56		12 M.	450	1	99.7	1.22
				1 P. M.	500	1	99.8	1.22
				2 "	500	1	99.8	1.22
				3 "	500	1	99.8	1.22
				4 "	500	1	99.8	1.22
				5 "	500	1	99.8	1.22
74	5	13		6 "	500	1	99.8	1.22
				7 "	500	1	99.8	1.30
				8 "	450	1	99.7	1.30
				9 "	450	1	99.7	1.30
				10 "	450	1	99.7	1.30
75	5	32		11 "	450	1	99.7	1.30
				12 "	450	1	99.7	1.30
			Jan. 29	1 A.M.	450	1	99.7	1.30
				2 "	450	1	99.7	1.30
				3 "	450	1	99.7	1.40
				4 "	450	1	99.7	1.40
76	5	19		5 "	450	1	99.7	1.40
				6 "	450	1	99.7	1.40
				7 "	400	1	99.7	1.40
				8 "	400	1	99.7	1.40
				9 "	300	1	99.7	1.40
77	6	50		10 "	300	0	100	1.30
				11 "	300	0	100	1.30
				12 M.	300	0	100	1.30
				1 P.M.	300	0	100	1.30
				2 "	275	0	100	1.30
				3 "	275	0	100	1.20
				4 "	275	0	100	1.20
				5 "	275	0	100	1.20
78	8	05		6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 "	250	0	100	1.12
			Jan. 30	1 A.M.	250	0	100	1.12
79	10	59		2 "	250	0	100	1.12
				3 "	250	0	100	1.12
				4 "	250	0	100	1.12
				5 "	250	0	100	1.12
				6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 M.	250	0	100	1.12
80	17	00		1 P.M.	200	0	100	1.12
				2 "	200	0	100	1.12
				3 "	200	0	100	0.99

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
80	17	00	Jan. 30	4 P. M.	200	0	100	0.99
				5 "	200	0	100	0.99
				6 "	200	0	100	0.99
				7 "	200	0	100	0.99
				8 "	200	0	100	0.99
				9 "	200	0	100	0.99
				10 "	200	0	100	0.99
				11 "	200	0	100	0.99
				12 "	200	0	100	0.99
				1 A.M.	200	0	100	0.99
				2 "	200	0	100	0.99
				3 "	200	0	100	0.99
				4 "	150	0	100	0.99
				5 "	150	0	100	0.99
81	13	40		6 "	150	0	100	0.99
				7 "	150	0	100	0.99
				8 "	100	0	100	0.63
				9 "	100	0	100	0.63
				10 "	100	0	100	0.63
				11 "	100	0	100	0.63
				12 M.	75	0	100	0.55
				1 P.M.	75	0	100	0.55
				2 "	75	0	100	0.55

Four of the runs included in Table 4 were too short, and, with the experience since gained in handling the plant, particularly the coagulant, could have been extended by an hour or two without difficulty.

The complete records of the operation of Roughing Filter No. 1 by runs, from September 12th, 1908, to February 3d, 1909, are given in Table 5.

Analyses.—There is no laboratory for chemical and bacteriological work at the plant, but an arrangement has been made by which samples of the raw water, the rough-filtered water, the final-filtered water, and water secured from a faucet in the borough, are collected and examined by the chemist and bacteriologist of the Harrisburg Filter Plant once each week, and at such other times as desired. Two plates, for total numbers of bacteria, are prepared from each sample, and presumptive tests are made for the presence of *B. Coli* in the raw- and filtered-water samples, five 1-cu. cm. sowings usually being made of each sample; the tests for turbidity, free CO_2, and alkalinity are made at the plant by the superintendent, who is a skilled chemist.

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation:		Loss of head:		Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million.			Bacteria per cubic centimeter.			Coagulant, in grains per gallon (average).
			Hours.	Minutes.	Initial.	Final.			Raw water.	Effluent.	Percentage removed.	Raw water.	Effluent.	Percentage removed.	
36	Jan. 10	Jan. 11	6	02	0.50	1.60	780 000	84.3	400	19.0	95	.....	....	..	....
37	" 11	" 11	4	21	0.50	1.58	890 000	102.0	400	20.0	95	18 200	6 000	68	0
38	" 11	" 11	5	18	0.48	1.95	810 000	92.9	400	28.0	93	.....	....	..	0
39	" 11	" 11	4	00	0.52	1.96	810 000	92.9	400	30.0	92	.....	....	..	0
40	" 11	" 11	3	11	0.55	1.95	880 000	100.8	400	30.0	92	.....	....	..	0.8
41	" 11	" 12	2	58	0.83	1.65	860 000	98.5	450	19.0	96	.....	....	..	0.9
42	" 12	" 12	3	40	0.60	2.15	750 000	84.9	450	13.0	97	.....	....	..	0.9
43	" 12	" 12	4	09	0.45	2.15	800 000	91.7	500	9.0	98	.....	....	..	0.9
44	" 12	" 12	2	44	0.60	1.55	890 000	102.0	500	6.0	98	.....	....	..	0.9
45	" 12	" 12	2	38	0.65	1.15	820 000	94.0	500	5.0	99	.....	....	..	0.9
46	" 12	" 12	3	43	0.50	1.90	830 000	95.0	500	6.0	98	.....	....	..	0.9
47	" 12	" 12	4	02	0.60	1.87	720 000	82.5	500	9.0	98	.....	....	..	0.9
48	" 12	" 13	8	01	0.74	1.55	770 000	88.2	450	7.0	98	.....	....	..	0.9
49	" 13	" 14	27	57	0.45	1.55	760 000	87.0	450	24.0	95	.....	....	..	0.3
50	" 14	" 15	12	05	0.52	1.74	770 000	88.2	300	5.0	98	.....	....	..	0.40
51	" 15	" 15	9	01	0.60	1.60	790 000	90.7	150	4.0	97	32 000	6 200	80	0.38
52	" 15	" 15	9	45	0.45	1.75	790 000	90.7	150	4.0	97	.....	....	..	0.33
53	" 15	" 16	4	40	0.55	1.22	760 000	87.1	125	4.0	97	.....	....	..	0.25
54	" 16	" 20	83	39	0.57	1.46	750 000	86.0	50	4.0	92	.....	....	..	....
55	" 20	" 23	68	14	0.48	1.60	720 000	82.5	20	3.0	85	5 975	1 445	76	0.10
56	" 23	" 24	17	38	0.62	1.45	730 000	83.7	15	1.0	93	.....	....	..	0.25
57	" 24	" 25	19	13	0.50	1.29	660 000	75.7	40	2.0	95	.....	....	..	0.25
58	" 25	" 26	9	58	0.38	1.75	630 000	72.2	560	1.0	99	.....	....	..	0.45
59	" 26	" 26	4	16	0.40	1.60	680 000	78.0	1 440	1.0	99	.....	....	..	2.07
60	" 26	" 26	3	02	0.40	2.10	780 000	89.4	1 200	1.0	99	.....	....	..	2.08
61	" 26	" 26	2	31	0.95	2.25	1 000 000	114.7	900	1.0	99	.....	....	..	2.00
62	" 26	" 26	2	26	0.95	1.96	840 000	96.2	700	1.0	99	.....	....	..	1.50
63	" 26	" 27	4	35	0.54	2.25	650 000	75.7	600	1.0	99	.....	....	..	1.58
64	" 27	" 27	2	58	0.57	1.98	860 000	98.5	500	2.0	99	.....	....	..	0.75
65	" 27	" 27	2	11	0.45	2.32	910 000	104.3	650	2.0	99	13 160	1 025	92	0.67
66	" 27	" 27	3	18	0.44	2.20	750 000	86.1	750	2.0	99	.....	....	..	0.63
67	" 27	" 27	3	17	0.40	1.75	810 000	92.9	600	2.0	99	.....	....	..	0.63
68	" 27	" 27	4	15	0.40	1.89	760 000	87.1	550	3.0	99	.....	....	..	0.63
69	" 27	" 28	3	53	0.45	2.20	830 000	95.2	525	4.0	99	.....	....	..	1.19

TABLE 5.—Summary of Results of Operation of Roughing Filter No. 1, by Runs.

Run No.	From:	To:	Time in operation: Hours.	Minutes.	Loss of head: Initial.	Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million. Raw water.	Effluent.	Percentage removed.	Bacteria per cubic centimeter. Raw water.	Effluent.	Percentage removed.	Coagulant, in grains per gallon (average).
1	Sept. 12	Oct. 5	570	43	0.35	1.76	710 000	81.4	6	....	....	.....	....	..	....
2	Oct. 5	" 19	334	00	0.53	1.18	707 000	81.0	2	0.5	75	.....	...	..	....
3	" 19	" 29	245	46	0.45	1.98	692 000	79.3	2	0.3	87	.....	....	..	....
4	" 29	Nov. 3	125	03	0.45	1.70	700 000	80.3	7	0.0	100	.....	....	..	....
5	Nov. 3	" 8	125	37	0.53	1.18	690 000	79.0	3	0.0	100	80	....	..	....
6	" 8	" 12	90	07	0.45	0.75	720 000	82.5	2	0.0	100	.....	....	..	....
7	" 12	" 16	95	23	0.54	0.85	685 000	78.5	2	0.0	100	.....	....	..	....
8	" 16	" 20	91	46	0.55	1.04	760 000	87.1	2	0.0	100	119	76	39	....
9	" 20	" 28	186	41	0.51	1.32	700 000	80.3	2	0.0	100	214	12	92	....
10	" 28	Dec. 4	161	00	0.45	1.68	770 000	88.2	1	0.0	100	.....	....	..	....
11	Dec. 4	" 8	102	55	0.60	1.08	770 000	88.2	1	0.0	100	408	310	24	....
12	" 8	" 14	136	31	0.88	1.06	800 000	91.7	2	0.0	100	220	125	43	....
13	" 14	" 18	94	41	0.58	1.04	740 000	84.3	5	1.0	80	1 680	680	60	....
14	" 18	" 20	50	10	0.40	1.00	690 000	79.0	7	1.0	87	.....	....	..	....
15	" 20	" 22	51	09	0.38	1.50	705 000	80.8	9	3.0	66	4 425	800	80	.. ..
16	" 22	" 24	39	02	0.60	1.61	750 000	86.0	15	3.0	80	.....	....	..	....
17	" 24	" 28	94	56	0.65	1.30	760 000	87.1	7	3.0	60	.....	....	..	....
18	" 28	Jan. 1	87	17	0.56	0.92	750 000	86.0	4	1.0	75	.....	....	..	....
19	Jan. 1	" 3	53	54	0.75	1.48	750 000	86.0	3	1.0	66	5 800	1 125	80	....
20	" 3	" 6	76	52	0.62	1.58	790 000	90.6	4	1.0	75	....	....	..	....
21	" 6	" 7	10	16	0.58	1.04	840 000	96.3	10	4.0	60	.....	....	..	....
22	" 7	" 8	13	38	0.42	1.02	810 000	92.9	100	4.0	96	.....	....	..	....
23	" 8	" 8	6	45	0.60	1.30	870 000	99.7	225	8.0	96	.....	....	..	....
24	" 8	" 8	5	39	0.48	1.30	860 000	98.6	300	8.0	97	.....	....	..	....
25	" 8	" 8	9	57	0.48	1.56	760 000	87.1	350	16.0	96	.....	....	..	....
26	" 8	" 9	7	56	0.62	1.55	830 000	95.1	400	14.0	97	.....	....	..	....
27	" 9	" 9	4	05	0.55	1.10	850 000	97.4	425	19.0	95	.....	....	..	....
28	" 9	" 9	3	32	0.57	1.45	860 000	98.6	450	21.0	93	.....	....	..	1.3
29	" 9	" 9	4	22	0.60	1.53	840 000	96.3	500	3.0	99	.....	....	..	0.6
30	" 9	" 9	3	57	0.67	1.63	870 000	99.7	550	3.0	99	.....	....	..	0.35
31	" 9	" 10	3	37	0.53	1.42	850 000	97.4	450	6.0	98	.....	....	..	0.3
32	" 10	" 10	5	07	0.54	1.80	810 000	92.9	450	15.0	96	.....	....	..	0.3
33	" 10	" 10	4	57	0.35	1.50	850 000	97.4	400	33.0	92	.....	....	..	0.3
34	" 10	" 10	3	51	0.56	1.75	890 000	102.0	400	13.0	97	.....	....	..	0.3
35	" 10	" 10	5	14	0.54	1.70	800 000	91.7	400	18.0	96	.....	...	..	0.3

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation: Hours.	Time in operation: Minutes.	Loss of head: Initial.	Loss of head: Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million. Raw water.	Silica turbidity, in parts per million. Effluent.	Silica turbidity, in parts per million. Percentage removed.	Bacteria per cubic centimeter. Raw water.	Bacteria per cubic centimeter. Effluent.	Bacteria per cubic centimeter. Percentage removed.	Coagulant, in grains per gallon (average).
1	Jan. 28	Jan. 28	3	09	0.60	2.17	760 000	87.1	500	1.0	99	.....	....	..	1.50
2	" 28	" 28	4	03	0.40	2.20	760 000	87.1	500	1.0	99	.....	....	..	1.45
3	" 28	" 28	4	04	0.45	2.20	780 000	89.5	500	1.0	99	.....	....	..	1.32
4	" 28	" 28	4	56	0.60	2.16	780 000	89.5	500	1.0	99	.....	....	..	1.32
5	" 28	" 28	5	13	0.50	1.82	750 000	86.1	450	1.0	99	.....	....	..	1.26
6	" 29	" 29	3	32	0.40	2.11	720 000	82.6	450	1.0	99	.....	....	..	1.33
7	" 29	" 29	5	19	0.57	1.99	740 000	84.9	300	0.0	100	.....	....	..	1.44
8	" 29	" 29	6	50	0.57	1.99	700 000	80.3	250	0.0	100	.....	....	..	1.26
9	" 30	" 30	8	05	0.40	1.88	710 000	81.4	250	0.0	100	.....	....	..	1.12
10	" 30	" 30	10	59	0.44	1.95	580 000	66.6	250	0.0	100	.....	....	..	1.12
11	" 30	" 31	17	00	0.44	1.86	560 000	64.2	250	0.0	100	.....	....	..	1.00
12	" 31	" 31	13	40	0.65	1.43	740 000	84.9	100	0.0	100	.....	....	..	0.65
13	" 31	Feb. 1	10	38	0.55	1.40	650 000	74.5	75	0.0	100	.....	....	..	0.55
14	Feb. 1	" 2	23	13	0.50	1.50	730 000	83.7	60	0.0	100	.....	....	..	0.50
15	" 2	" 3	12	47	0.60	1.86	810 000	92.9	50	0.0	100	.....	....	..	0.50
16	" 3	" 3	10	11	0.60	1.46	790 000	90.7	30	2.0	93	.....	....	..	....

The area of the filter = 0.008719 acre.

Air Wash.– The roughing filters, when clogged, or when not yielding a satisfactory effluent, are cleaned by washing with air and water, first shutting off the raw-water supply and then, if pushed for time, opening the sewer gate and wasting the water upon the surface of the filter down to the level of the wash-water troughs. The filter, in the meantime, being in operation, is allowed to run until the surface of the water on it has dropped about 6 in. below the edges of the wash-water troughs; the effluent gate is then closed, and compressed air is discharged into the underdrains from the air receiver. The compressed air is stored in the receiver at a pressure of approximately 100 lb. per sq. in. at the beginning of the wash, and is discharged through a reducing valve set at about 10 lb. on the filter side of the valve; this, however, does not represent the pressure at which the air is applied during the wash, as the 10 lb. is largely used up in friction through the small air pipes leading to the filters. The air issues into the filter through the orifices in the bottom of the 2-in. galvanized-iron underdrain pipes 1 in. above the floor of the filter. The bottoms of the pipes lying all in the same plane, the air issues from the orifices only as it is forced out by the pressure in the delivery pipe, its tendency to issue by its own buoyancy being overcome by placing the orifices in the bottoms of the pipes; within limits, therefore, it is possible to deliver to the filter any quantity of air desired, and to have it issue through all the orifices in the underdrain pipes with practical uniformity. The application of the air is continued until the contents of the receiver, approximating 1 500 cu. ft. of free air, is discharged, which takes from 4 to 5 min., as the reducing valve is now set, and this corresponds to approximately 1 cu. ft. of free air per square foot of filter surface per minute. Apparently, this is sufficient, for, as far as can be ascertained, there has been no tendency for suspended matters to accumulate in the filter permanently. The objection to the use of a large quantity of air, or to very heavy air washing, is the likelihood of the disturbance of the gravel underdrains to be followed by the loss of some of the filtering material; and the tendency to aid in producing vertical stratification in the filter, in connection with the water-wash, allowing some portions to clog up completely, and increasing the velocity of filtration through the more porous parts to such an extent as to render the filter ineffective. In order to guard against this tendency, the superintendent, from time to time, stirs up by hand the

filtering materials in the roughing filters, using a rake while the wash-water is being applied; this dissipates effectively any tendency of the materials to collect in vertical strata, and maintains the porosity and efficiency of the filter.

Water-Wash.—After the air is turned off the filters are washed with filtered water from the street mains. The water is applied through the underdrains, and the dirty water overflows into the wash-water troughs and passes thence to the river through the sewer. The wash-water is ordinarily applied at a rate equivalent to about three and one-half times the rate of filtration, or between 8 and 9 vertical inches in the filter per minute, the duration of the water-wash being from 5 to 6 min., and the quantity of water used per wash about 10 000 gal. In addition to that used for washing, about 5 000 gal. are wasted from the surface of the filter at each wash when the turbidity is high and the filters have to be washed frequently. It cannot, as yet, be told with accuracy just what percentage of wash-water will be required during the year, but a prediction of the quantity can be made from data secured at the Harrisburg Testing Station, and at the Harrisburg Filter Plant. The number of washes required will depend on the turbidity of the raw water, as this causes the clogging of the filter and necessitates the use of a coagulant.

TABLE 6.—Effect of Turbidity of Raw Water on Lengths of Runs of Roughing Filters at Steelton.

Turbidity of applied water, in parts per million.		Number of days per year when such turbidities may be expected to prevail.	Average lengths of runs.
Range. (1)	Average. (2)	(3)	(4)
0- 50	25	160	6 days.
51- 75	62	65	24 hours.
76- 100	88	35	17 "
101- 150	125	29	12 "
151- 200	175	23	10 "
201- 250	225	13	9 "
251- 500	375	25	7 "
501- 800	650	7	5 "
801-1 200	1 000	5	4 "
1 200 +	1 400	3	3 "

Lengths of Runs.—The data thus far secured from the Steelton plant, in addition to information from the Harrisburg Testing Station, indicate that the lengths of runs from the roughing filters, when operat-

ing at a rate of from 75 000 000 to 125 000 000 gal. per acre per day, would be about as shown in Table 6. Column 1 indicates the range of turbidity of the raw water, Column 2 the average turbidity, Column 3 the approximate number of days per year when such turbidities may be expected to prevail, and Column 4 the lengths of runs of the roughing filters.

Using the data in Table 6, 428 washes per year per filter will be required, calling for a total of about 6 700 000 gal. of wash- and waste-water, assuming that the water on the surface of the roughing filters is wasted at each wash. As it takes about 15 min. to wash a filter, the necessary lost time for the 428 washes would be 4½ days per year, and the output of each filter for the 360.5 days of operation, at its full rated capacity, 1 500 000 gal. per day, would be 540 500 000 gal.; the wash- and waste-water, on this basis, would represent 1.2% of the net output of the filter for the year.

The longest run yet experienced was about 24 days, and the shortest 2 hours 11 min. It is believed that with more experience the shortest runs need not be less than about 4 hours.

TABLE 7.—AVERAGE DOSE OF COAGULANT.

TURBIDITY OF APPLIED WATER, IN PARTS PER MILLION.		Number of days per year when such turbidities may be expected to prevail.	Average dose of aluminum sulphate, in grains per gallon.	Average dose of lime, in grains per gallon.
Range. (1)	Average. (2)	(3)	(4)	(5)
0– 50	25	160	0.0	0
51– 75	62	65	0.4	0
76– 100	88	35	0.5	0
101– 150	125	29	0.6	0
151– 200	175	23	0.7	0
201– 250	225	13	0.8	0
251– 500	375	25	1.0	0.25
501– 800	650	7	1.4	0.40
801–1 200	1 000	5	1.7	0.60
1 200+	1 400	3	2.1	0.75

Average Dose of Coagulant.—From the data thus far secured, it would appear that the average quantities of coagulant necessary in the operation of the plant would be about as shown in Table 7, from which a calculation indicates that the average dose required through a series of years would be equivalent to 0.37 gr. per gal., or 53 lb. per million gallons of water treated, which, at $20 per ton, gives the average cost for coagulant as 53 cents per million gallons. For the

first year the quantity required will fall far short of this figure, the average dose from September 16th, 1908, to April 18th, 1909, having been but 0.22 gr. per gal.

Effect of Turbidity on Initial Loss of Head.—The initial loss of head, at the commencement of operation of a roughing filter, is not affected either by the turbidity of the raw water or the quantity of aluminum sulphate used, the effects of these two factors being evident only in the rate at which the loss of head increases, and consequently in the lengths of the runs.

Effect of Rate on Quantity Filtered.—The rate of filtration affects only slightly the total quantity of water that may be filtered between washings of the roughing filters; whatever slight effect there may be apparently indicates that somewhat larger quantities can be filtered between washings at high rates than at low rates, but, of course, at the expense of somewhat more frequent washings.

Operation of Slow Filters.

The slow filters are operated on the continuous plan, the rough-filtered water being controlled so as to stand about 2 ft. deep on the surface of the slow filters.

The method of regulating the rate of filtration need not be again described, it being sufficient to state that the yield of the slow filters is responsive to the draft of the municipal high-pressure pumps, up to the limit of the discharging capacity of the regulating devices on the filters. As filtration proceeds, the slow filters gradually clog up, and the loss of head increases; provision is made to allow for a total loss of head of 7 ft. in the slow filters, which is the full depth of the filtering materials, and the superincumbent water.

Daily Records.—The slow filters were put in operation on September 16th, 1908. The daily records of the operation of Filter No. 1 are given in Table 8, from which it will be seen that the accumulation of the loss of head proceeded with comparative slowness during September, October, and November, while during December the occasional increase in turbidity of the raw water began to have its effect on clogging, bringing the total loss of head up to about 2 ft. on the last day of December. In January, 1909, during which there were two periods of bad water, the loss of head increased to about 6 ft., a gain of 4 ft., during the month, and it became necessary to start scraping.

On the morning of January 30th, therefore, the supply to Filter No. 1 was shut off, the water was drained down to the surface of the filter, and filtration was continued for a short time to allow the water to fall a sufficient distance below the sand surface.

TABLE 8.—Record of Operation of Slow Filter No. 1, September 16th, 1908, to February 1st, 1909.

Date.	Time in service: Days.	Hours.	Loss of head, in feet.	Rate of Operation: Gallons daily.	Millions of gallons per acre per day.
(1)	(2)		(3)	(4)	(5)
Sept. 16	..	..	0.38	680 000	4.7
" 17	1	..	0.36	650 000	4.5
Oct. 1	15	..	0.33	610 000	4.2
" 4	18	..	0.35	600 000	4.1
" 9	23	..	0.45	600 000	4.1
" 10	24	..	0.30	520 000	3.6
" 14	28	..	0.49	510 000	3.5
" 17	31	..	0.35	500 000	3.4
" 23	37	..	0.40	530 000	3.7
" 28	42	..	0.35	630 000	4.3
" 31	45	..	0.30	520 000	3.6
Nov. 4	49	..	0.40	580 000	4.0
" 14	59	..	0.63	500 000	3.4
" 25	70	..	0.61	500 000	3.4
" 30	75	..	0.75	500 000	3.4
Dec. 1	76	..	0.75	500 000	3.4
" 5	80	..	0.85	500 000	3.4
" 8	83	..	1.70	500 000	3.4
" 11	86	..	1.50	680 000	4.7
" 17	92	..	1.20	500 000	3.4
" 23	98	..	1.60	500 000	3.4
" 26	101	..	1.93	500 000	3.4
" 27	102	..	1.95	500 000	3.4
" 31	106	..	2.07	500 000	3.4
Jan. 1	107	..	2.09	500 000	3.4
" 3	109	..	2.20	500 000	3.4
" 4	110	..	2.35	500 000	3.4
" 5	111	..	2.42	500 000	3.4
" 9	115	..	2.92	500 000	3.4
" 12	118	..	3.70	500 000	3.4
" 15	121	..	3.74	500 000	3.4
" 18	124	..	4.23	500 000	3.4
" 22	128	..	4.12	500 000	3.4
" 25	131	..	5.01	500 000	3.4
" 27	133	..	5.35	500 000	3.4
" 28	134	..	5.99	500 000	3.4
" 29	135	..	5.75	500 000	3.4
" 30	135	7	6.01	500 000	3.4

Filter put in service September 16th, 1908.
Filter scraped January 30th, 1909.
Sand removed, washed, and stored, 18 cu. yd. = 124 cu. yd. per acre.
Quantity filtered during run = 71 000 000 gal.
Millions of gallons per acre filtered between scrapings = 491.
Average rate of filtration = 3 523 000 gal. per acre daily.
Net area of sand surface = 0.1446 acre.
Wasted from surface of filter for scraping .....104 500 gal.
Water drained out, and water used for refilling from below...156 800

261 300 = 0.36% of filtrate.

Scraping.—Filter No. 1 was scraped by the superintendent of the plant and a force of five men, the actual time occupied in piling up

the scrapings ready for transportation to the washer being 3 hours, or at the rate of about 7 sq. ft. of filter scraped per man per minute. The scraping on this filter was rather deep, averaging nearly an inch, and being less than ¾ in. in very few places. A portion of the extra scraping was made necessary by the accidental dropping of one of the manhole covers into the filter early in January. This caused a great disturbance of the sand, over an area of perhaps 100 sq. ft., as the cover turned when it struck the water and penetrated the sand bed on its edge, the lower corner having reached almost to the gravel underdrains. The cover was taken out, and all the discolored sand which had been driven down into the bed from the *schmützdecke* was dug out and sent to the washer.

Sand Transportation and Washing.—The ejector and washing plant worked satisfactorily as to the handling of the sand, but, as it had not been in use before, the men did not attain a high degree of efficiency in its operation. It is believed that greater familiarity with its use, and the proper regulation of the relative force of the different jets, will permit of handling the sand much more rapidly and efficiently than was done. The total quantity of sand removed from Filter No. 1 was about 18 cu. yd., or 124 cu. yd. per acre of filter surface, to transport which to the sand washer, wash it, and return it to the sand troughs on top of the filters occupied about 8 hours, one man attending to the sand washer, one to the hose returning the sand to the sand troughs, one raking the scraped surface of the filter, and an average of three shoveling the dirty sand to the portable ejector.

After the sand for one filter had all been washed, the portable ejector was taken apart and an irregularly-shaped stone, which had carelessly been left in one of the water mains, was found wedged in the nozzle. This had caused the deflection of the jet of water so that the throat on the discharge side had been cut deeply. After the throat had been replaced with a new one, the original capacity of the ejector was restored, and the scraping and transporting of sand to the washer from the other filters proceeded with more expedition.

Ejector Nozzles.—The nozzle in the portable ejector throws a ⅝-in. jet of water into a throat with a 1½-in opening reduced to 1 in. in diameter 1½ in. from the front of the throat and increasing to 1⅝ in. in diameter at the back end of the throat 7 in. from the face; the nozzle and throat stand 1½ in. apart. The nozzle reduces from 2 in. in

diameter to the ¾-in. jet in a length of about 6 in. The nozzles and throats are of chilled cast iron.

The nozzles for the sand washer have a $\frac{13}{16}$-in. jet, and the throats reduce from 1½ in. to 1¼ in. in diameter in 1½ in., increasing to 1¾ in. in 5½ in. The portable ejector and the washing hoppers were supplied by the Norwood Engineering Company, from designs by Allen Hazen, M. Am. Soc. C. E., originally worked out for the filter plant at Washington, D. C., and containing recent improvements suggested by Mr. Hazen. The washing plant consists of only two hoppers, and practically all the washing is done in the first one, the sand as it is thrown to the second hopper from the first being sufficiently clean to be used in the filter. The wash-water overflows the second hopper with very little turbidity.

The lift from the portable ejector to the top of the pipe discharging into the washing hopper is about 15 ft.; from the second washing hopper to the sand troughs there is a drop of about 6 ft., so that comparatively little pressure is required to transport the sand from the sand washer back to the sand bins.

The washing hoppers are provided with auxiliary jets at the bottom to compensate for the quantity of dirty water actually picked up by the jet and forced through to the succeeding hopper, the auxiliary jet also serving the purpose of stirring up the sand at the bottom of the hopper so as to facilitate its transportation by the water jet. The dirty water overflowing the washing hoppers passes through a reinforced concrete box in order to catch such sand as may escape with the wash-water; the dirty water escapes from this box to the sewer by an overflow.

The water which transports the washed sand to the sand troughs overflows a weir at one end of each sand box and escapes to the sewer.

Refilling after Scraping.—After the dirty sand is all thrown out of the filter and the surface of the clean sand has been raked over to remove footprints and give a smooth even surface, filtered water is admitted through the underdrains until the surface of the filter sand is covered about 2 in. in depth; the raw water is then admitted, gently at first, and the filter is refilled to its proper operating depth with the rough-filtered water.

Resumption of Filtration.—In starting this filter in operation after scraping, the effluent control valve was regulated so that the filter would

deliver, according to its rate-gauge, 800 000 gal. a day, it being necessary to choke down the discharge of the clean filter to a proper rate because the two other filters were very much clogged up and could not yield their proportional part of the water without a loss of head of 5 or 6 ft.; with such a loss of head the clean filter would at once start off with its maximum allowable rate, which was deemed inadvisable, the filter being so new, and having been started in operation so late in the season.

Lengths of Runs.—Filter No. 2 was scraped on February 2d and was put back in service on February 3d, and Filter No. 3 on February 4th, 1909. The lengths of runs of these three filters, therefore, were 135.5, 137.5 and 140.5 days, respectively. Filter No. 1 delivered during its run 71 000 000 gal. at an average rate of 3 623 000 gal. per acre daily, the total yield corresponding to 491 000 000 gal. per acre between scrapings. The figures for the other two filters were a trifle larger.

The total quantity of water wasted from the surface of the filter prior to scraping, and the quantity used for refilling the filter from below, before starting in operation again, corresponded to 0.14 of 1% of the quantity of water filtered.

The three slow filters have now (May 10th, 1909) been in service for 14 weeks since the last (and only) scraping, and the loss of head is but 0.7 ft.

Efficiency of the Plant.

The efficiency of the entire plant, in the removal of turbidity and bacteria, is exhibited in the daily records given in Table 9, which date from November 1st, 1908, for the reason that no bacterial analyses were made prior to November 5th.

The efficiency in the removal of turbidity has been 100 per cent. During November and December, when the turbidity of the raw water was low and the bacteria were not numerous, the effluent water was satisfactory. The same is true during January, with the exception of the second week when, as has already been explained, the superintendent was deceived by the deterioration of his silica-turbidity standards, and the quantity of coagulant was inadequate to produce a satisfactory effluent. The results subsequent to that date are good. The samples collected on the afternoon of February 5th, when Filter No. 3 was being scraped, represent the bacterial contents of the effluents of Filters Nos. 1 and 2, which had been scraped, No. 2 within two days

TABLE 9.—Daily Efficiency of Plant in Removal of Turbidity and Bacteria.

Date.	Average turbidities, in parts per million: Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Bacteria, per cubic centimeter: Raw water.	Rough filtered water.	Filtered water.	Percentage of removal.	Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
1908.										
Nov. 1...	5	0	0	100	.....	.....	.....	.....	.....	.....
2...	4	0	0	100	.....	.....	.....	.....	.....	.....
3...	3	0	0	100	.....	.....	.....	.....	.....	.....
4...	2	0	0	100	.....	.....	.....	.....	.....	.....
5...	3	0	0	100	80	.....	3	96	.....	.....
6...	3	0	0	100	.....	.....	.....	.....	.....	.....
7...	3	0	0	100	.....	.....	.....	.....	.....	.....
8...	3	0	0	100	.....	.....	.....	.....	.....	.....
9...	2	0	0	100	.....	.....	.....	.....	.....	.....
10...	2	0	0	100	.....	.....	.....	.....	.....	.....
11...	2	0	0	100	.....	.....	.....	.....	.....	.....
12...	2	0	0	100	.....	.....	.....	.....	.....	.....
13...	2	0	0	100	.....	.....	.....	.....	.....	.....
14...	2	0	0	100	.....	.....	.....	.....	.....	.....
15...	1	0	0	100	.....	.....	.....	.....	.....	.....
16...	2	0	0	100	.....	.....	.....	.....	.....	.....
17...	3	1	0	100	.....	.....	.....	.....	.....	.....
18...	3	1	0	100	.....	.....	.....	.....	.....	.....
19...	2	0	0	100	.....	.....	.....	.....	.....	.....
20...	2	0	0	100	119	76	6	95	.....	.....
21...	2	0	0	100	.....	.....	.....	.....	.....	.....
22...	2	0	0	100	.....	.....	.....	.....	.....	.....
23...	2	0	0	100	.....	.....	.....	.....	.....	.....
24...	2	0	0	100	.....	.....	.....	.....	.....	.....
25...	2	0	0	100	.....	.....	.....	.....	.....	.....
26...	1	0	0	100	.....	.....	.....	.....	.....	.....
27...	1	0	0	100	214	12	3	99	.....	.....
28...	1	0	0	100	.....	.....	.....	.....	.....	.....
29...	1	0	0	100	.....	.....	.....	.....	.....	.....
30...	1	0	0	100	.....	.....	.....	.....	.....	.....
Dec. 1...	1	0	0	100	.....	.....	.....	.....	.....	.....
2...	1	0	0	100	.....	.....	.....	.....	.....	.....
3...	1	0	0	100	.....	.....	.....	.....	.....	.....
4...	1	0	0	100	.....	.....	.....	.....	.....	.....
5...	1	0	0	100	.....	.....	.....	.....	.....	.....
6...	1	0	0	100	168	310	52	87.2	.....	.....
7...	1	0	0	100	.....	.....	.....	.....	.....	.....
8...	1	0	0	100	.....	.....	.....	.....	.....	.....
9...	1	0	0	100	.....	.....	.....	.....	.....	.....
10...	1	0	0	100	.....	.....	.....	.....	.....	.....
11...	1	1	0	100	.....	.....	.....	.....	.....	.....
12...	2	1	0	100	220	125	8	96.4	.....	.....
13...	3	1	0	100	.....	.....	.....	.....	.....	.....
14...	4	1	0	100	.....	.....	.....	.....	.....	.....
15...	4	1	0	100	.....	.....	.....	.....	.....	.....
16...	4	1	0	100	.....	.....	.....	.....	.....	.....
17...	4	0	0	100	1 680	680	71	95.9	.....	.....
18...	7	1	0	100	.....	.....	.....	.....	.....	.....
19...	8	1	0	100	.....	.....	.....	.....	.....	.....
20...	4	1	0	100	.....	.....	.....	.....	.....	.....
21...	5	1	0	100	4 425	800	58	98.7	.....	.....
22...	15	4	0	100	.....	.....	.....	.....	.....	.....
23...	18	2	0	100	.....	.....	.....	.....	.....	.....

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.		
1908.										
Dec. 24...	10	3	0	100	.....	.....	.....	.....	.....	.....
25...	10	3	0	100	.....	.....	.....	.....	.....	.....
26...	6	2	0	100	.....	.....	.....	.....	.....	.....
27...	5	1	0	100	.....	.....	.....	.....	.....	.....
28...	4	1	0	100	.....	.....	.....	.....	.....	.....
29...	3	1	0	100	.....	.....	.....	.....	.....	.....
30...	3	1	0	100	.....	.....	.....	.....	.....	.....
31...	2	1	0	100	.....	.....	.....	.....	.....	.....
1909.										
Jan. 1...	4	1	0	100	5 800	1 125	100	98.3	.....	.....
2...	2	1	0	100	.....	.....	.....	.....	.....	.....
3...	3	1	0	100	.....	.....	.....	.....	.....	.....
4...	3	1	0	100	.....	.....	.....	.....	.....	.....
5...	3	1	0	100	....	.....	.....	.....	.....	.....
6...	7	2	0	100	.....	.....	.....	.....	.....	.....
7...	120	4	0	100	.....	.....	.....	.....	.....	.....
8...	250	10	0	100	.....	.....	.....	.....	.....	.....
9...	450	14	0	100	.....	.....	.....	.....	0.5	.....
10...	400	20	0	100	.....	.....	.....	.....	0.15	.....
11...	425	20	0	100	18 200	6 000	1 105	93.9	0.30	.....
12...	450	7	0	100	.....	.....	.....	.....	0.90	.....
13...	250	12	0	100	.....	.....	.....	.....	0.10	.....
14...	180	12	0	100	.....	.....	.....	.....	0.20	.....
15...	125	4	0	100	32 000	6 200	320	99.0	0.35	.....
16...	75	4	0	100	.....	.....	.....	.....	0.30	.....
17...	60	5	0	100	.....	.....	.....	.....	0.10	0
18...	30	4	0	100	.....	.....	.....	.....	.....	.....
19...	25	4	0	100	.....	.....	.....	.....	.....	.....
20...	20	4	0	100	.....	.....	.....	.....	.....	.....
21...	35	4	0	100	5 975	1 445	115	98.1	.....	.....
22...	15	2	0	100	.....	.....	.....	.....	0.25	.....
23...	10	1	0	100	.....	.....	.....	.....	0.25	.....
24...	20	1	0	100	.....	.....	.....	.....	0.25	18
25...	50	2	0	100	.....	.....	.....	.....	0.25	33
26...	770	1	0	100	.....	.....	.....	.....	1.38	36
27...	600	2	0	100	13 160	1 025	11	99.9	0.78	33
28...	490	1	0	100	.....	.....	.....	.....	1.32	16
29...	325	0	0	100	.....	.....	.....	.....	1.35	32
30...	225	0	0	100	.....	.....	.....	.....	1.00	20
31...	100	0	0	100	.....	.....	.....	.....	0.65	32
Feb. 1...	75	0	0	100	.....	.....	.....	.....	0.55	16
2...	60	0	0	100	.....	.....	.....	.....	0.50	18
3...	20	3	1	95	.....	.....	.....	.....	.....	14
4...	20	2	0	100	.....	.....	.....	.....	0.25	.....
5*..	15	0	0	100	3 025	295	188	94	0.25	.....
6...	70	0	0	100	.....	.....	.....	.....	0.25	14
7...	45	0	0	100	.....	.....	.....	.....	0.40	17
8...	190	2	0	100	.....	.....	.....	.....	0.60	14
9...	170	0	0	100	.....	.....	.....	.....	0.60	12
10...	215	0	0	100	.....	.....	.....	.....	0.60	13
11...	170	0	0	100	.....	.....	.....	.....	0.60	12

* Filter No. 3 out of service for scraping; Filter No. 1 was scraped on January 30th, and No. 2 on February 3d.

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.	Free CO_2 in raw water, in parts per million.	Average dose of lime used, in grains per gallon.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.				
1909.												
Feb. 12..	270	0	0	100	16 000	500	47	99.7	0.65	13	....	0
13..	160	0	0	100	....	....	..	....	0.65	15	....	0
14..	80	0	0	100	....	....	..	....	0.35	16	....	0
15..	50	0	0	100	....	....	..	....	0.33	14	....	0
16..	120	0	0	100	....	....	..	....	0.50	13	....	0
17..	200	0	0	100	....	....	..	....	0.70	10	....	0
18..	310	8	0	100	....	....	..	....	0.80	6	3	0.42
19..	190	0	0	100	4 500	115	25	99.5	1.00	6.5	3.5	0.42
20..	125	2	0	100	....	....	..	....	1.20	7.5	3	0.30
21..	300	0	0	100	....	....	..	....	0.90	9	4	0.30
22..	390	0	0	100	....	....	..	....	0.80	9	....	0
23..	465	2	0	100	....	....	..	....	1.10	11	....	0
24..	380	0	0	100	....	....	..	....	1.00	11.5	....	0
25..	275	0	0	100	....	....	..	....	0.85	11.5	....	0
26..	310	0	0	100	....	....	..	....	0.80	10	....	0
27..	310	0	0	100	6 850	70	10	99.8	0.80	11	....	0
28..	260	0	0	100	....	....	..	....	0.80	11	....	0
Mch. 1..	150	0	0	100	....	....	..	....	0.65	11	....	0
2..	120	0	0	100	....	....	..	....	0.65	8	....	0
3..	100	0	0	100	....	....	..	....	0.50	7	....	0
4..	65	0	0	100	....	....	..	....	0.40	6	....	0
5..	52	1	0	100	....	....	..	....	0.40	6	....	0
6..	47	1	0	100	3 425	750	61	98.2	0.00	8	....	0
7..	45	0	0	100	....	....	..	....	0.00	10	....	0
8..	60	0	0	100	....	....	..	....	0.35	11	....	0
9..	60	0	0	100	....	....	..	....	0.35	12	....	0
10..	50	0	0	100	....	....	..	....	0.35	12	....	0
11..	50	0	0	100	....	....	..	....	0.35	11	....	0
12..	90	0	0	100	....	....	..	....	0.35	11	....	0
13..	120	0	0	100	2 325	85	10	99.6	0.40	14	....	0
14..	115	0	0	100	....	....	..	....	0.40	16	....	0
15..	100	0	0	100	....	....	..	....	0.40	14	....	0
16..	75	0	0	100	....	....	..	....	0.30	14	....	0
17..	47	0	0	100	....	....	..	....	0.30	12	....	0
18..	42	0	0	100	....	....	..	....	0.30	14	....	0
19..	45	0	0	100	....	....	..	....	0.30	14	....	0
20..	40	0	0	100	....	....	..	....	0.30	14	....	0
21..	30	0	0	100	....	....	..	....	0.25	14	....	0
22..	25	0	0	100	....	....	..	....	0.25	16	....	0
23..	22	0	0	100	1 000	55	5	99.5	0.25	16	....	0
24..	20	0	0	100	....	....	..	....	0.00	15	....	0
25..	20	0	0	100	....	....	..	....	0.25	15	....	0
26..	25	0	0	100	....	....	..	....	0.25	14	....	0
27..	110	0	0	100	*	220	17	....	0.40	14	....	0
28..	140	0	0	100	....	....	..	....	0.55	14	....	0
29..	120	0	0	100	....	....	..	....	0.57	12	....	0
30..	150	0	0	100	....	....	..	....	0.62	14	....	0
31..	135	0	0	100	....	....	..	....	0.57	14	....	0
Apr. 1..	100	1	0	100	....	....	..	....	0.47	12	....	0
2..	85	0	0	100	....	....	..	....	0.45	14	....	0
3..	72	1	0	100	2 950	56	16	99.5	0.40	16	....	0
4..	60	1	0	100	....	....	..	....	0.32	16	....	0
5..	50	1	0	100	....	....	..	....	0.35	17	....	0
6..	50	1	0	100	....	....	..	....	0.40	19	....	0
7..	50	1	0	100	....	....	..	....	0.40	19	....	0
8..	55	0	0	100	....	....	..	....	0.45	18	....	0
9..	100	0	0	100	....	....	..	....	0.52	15	....	0
10..	95	0	0	100	....	....	..	....	0.48	14	....	0
11..	80	0	0	100	....	....	..	....	0.45	16	....	0
12..	95	0	0	100	....	....	..	....	0.50	15	....	0
13..	80	1	0	100	1 500	38	4	99.7	0.45	14	....	0
14..	42	1	0	100	....	....	..	....	0.35	14	....	0
15..	140	1	0	100	....	....	..	....	0.49	13	....	0
16..	345	1	0	100	....	....	..	....	0.80	12	....	0
17..	350	20	0	100	8 600	475	2	99.9	1.25	12	3	0.25
18..	340	0	0	100	....	....	..	....	1.00	11	2	0.25

* Raw water sample not taken.

and No. 1 within 5 days of the date of taking the sample. The individual samples of the effluents of Nos. 1 and 2 showed the following counts:

Filter No. 1, put in service, January 31st.
Sample No. 1..... 185 (collected February 5th)
Sample No. 2..... 170 (" " ")

Average..... 176

Filter No. 2, put in service February 3d.
Sample No. 1..... 200 (collected February 5th)
Sample No. 2..... 185 (" " ")

Average..... 198

A slight improvement shows in the effluent of the older filter.

The efficiency in the removal of *B. Coli* is shown in Table 10.

TABLE 10.—PRESUMPTIVE TESTS FOR PRESENCE OF *B. Coli* IN THE RAW WATER, FILTERED WATER AT THE FILTER PLANT, AND TAP WATER IN STEELTON, PA.

Date.	Raw Water.		Filtered Water.		Tap Water.	
	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.
Nov. 20, 1908.	5	3	5	0	5	0
" 27	5	3	5	0	5	0
Dec. 6	5	3	5	0	5	0
" 12	5	1	5	0	..	..
" 17	5	1	5	0	5	0
" 21	4	1	4	1	4	0
Jan. 1, 1909.	5	1	5	0	5	0
" 11	5	5	5	4	5	5
" 15	5	3	5	1	5	0
" 21	5	4	5	0	5	1
" 27	5	5	5	0	5	0
Feb. 5	4	3	4	0	4	0
" 12	4	4	4	0	4	0
" 19	3	1	3	0	3	0
" 27	5	5	5	1	5	0
Mch. 6	5	2	5	0	5	0
" 13	5	3	5	0	5	0
" 23	5	4	5	0	5	0
" 27	..	..	5	0	5	0
Apr. 3	5	5	5	0	5	0
" 13	4	3	4	0	4	0
" 17	4	4	4	0	4	0

The presence of *B. Coli* in the filtered water on December 21st, January 11th, and January 15th was in part due to lack of, or insufficient use of, coagulant in the preliminary process. The dropping of

a manhole cover into Filter No. 3 on January 1st, however, may have contributed to the deterioration of the effluent during the early part of that month.

Thus far, no chemical analyses have been made of the applied water and final filtrate at the Steelton plant, and no data, therefore, have been secured on the degree of nitrification accomplished, or on the other usual changes expected. It is evident that between periods of turbid water, when no coagulant is being used, the slow sand filters will be penetrated deeply by such suspended matters as may pass through the roughing filters, among which would be bacteria, and in this respect the work required of these filters will differ from that performed by the ordinary slow sand filter. During the past few months no visible suspended matters have been left in the rough-filtered water.

In nearly all plants using aluminum sulphate as a coagulant, trouble, either continuously or intermittently, has been experienced with discoloration of the water, generally only the hot water, by iron rust. This trouble has been more or less serious at Harrisburg, Pa., Watertown, N. Y., Charleston, S. C., Hackensack, N. J., and various other places. Thus far, there has been no trouble of this sort at Steelton, although at times as much as 2 gr. of aluminum sulphate has been used per gallon of water filtered.

Cost of Operation.—The plant has not yet been in operation sufficiently long to obtain accurate data, but an approximation for an average year would indicate an expense, per million gallons, when filtering 3 000 000 gal. of water daily, about as follows:

COST OF PURIFICATION OF STEELTON WATER PER MILLION GALLONS, ON A BASIS OF FILTERING 3 000 000 GAL. PER DAY.

Operation of Roughing Filters.

Labor	$1.55
Superintendence	0.42
Supplies and analyses of water	0.30
Coagulant	0.53
Power	0.08
Wash-water	0.13
Light	0.05
Coal	0.10
	$3.16

Operation of Slow Filters.

Scraping, 5 times per year	$0.60	
Transporting and washing sand	0.06	
Re-sanding filters	0.25	
Superintendence	0.42	
		$1.33
Total		$4.49

Interest and sinking fund charges are not included.

Acknowledgments.—The principal credit for the existence of the Steelton filter plant is due to J. V. W. Reynders, M. Am. Soc. C. E., President of Councils of Steelton, Chairman of the Filtration Committee of Councils and Vice-President of the Pennsylvania Steel Company, who, by arranging a well-conducted campaign of education and devoting much thought and time to the cause, prepared the way to enable Councils to see the necessity of making provisions for a purer water supply.

On its completion, the plant was turned over for operation to the Board of Water Commissioners, Mr. George H. Roberts, President, under the general direction of the Superintendent of the Water-Works, Mr. O. P. Baskin. The operation of the filters is in charge of Mr. M. B. Litch, as Filter Superintendent, the writer, acting in an advisory capacity, as required.

The works were built by contract under the supervision of the writer's engineering force, Mr. Paul Hooker being Principal Assistant during the designing of the works, and Resident Engineer during their construction; to his faithfulness the plant bears permanent witness.

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PAPERS AND DISCUSSIONS

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CHARACTERISTICS OF MODERN HYDRAULIC TURBINES.

By Chester W. Larner, Esq.

To be Presented October 20th, 1909.

The design and selection of turbines for hydraulic power-plants were at one time—and that within comparatively recent years—processes of an extremely uncertain nature. They partook quite largely of experimentation, much of which, having no sound basis of scientific knowledge, degenerated into the purest guesswork. During late years, however, the various problems involved have been the subject of much careful and intelligent study, and the situation has become decidedly better, although there is still room for vast improvement.

One of the chief obstacles in the path of progress at the present time is the scarcity of published data on the performance of turbines of recent construction. A large quantity of exceedingly valuable information has been accumulated by the manufacturers, but very little of it is available to the public in published form, or, in fact, in any form whatever. The publication of tests seems to be an idea altogether abhorrent to the majority of turbine builders in America. It is with this policy of reticence that the writer is inclined to take issue, not only because it is of an unprofessional character, but because

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

it defeats the very commercial ends which, according to some peculiar method of mistaken reasoning, it is supposed to further. It is only by a broad-minded policy of publicity on the part of the manufacturers, that the Engineering Profession at large can become acquainted with the possibilities of turbine construction, and thereby profit in the selection of this apparatus by the progress which is being made in the development of the art.

The publication of data relative to results which have been accomplished does not necessitate the publication of any information concerning the details of design on which these results depend. Any manufacturer is justified in protecting for his own use any points of superiority in design which he may be able to devise. His success depends on them, and he could not reasonably be expected to publish them for the benefit of his competitors; but the results which can be accomplished by his machines in operation are quite a different matter. This information is of vital importance to all engineers interested in the development of water-power, and it is in pursuance of this conviction that the writer has prepared this paper.

In the evolution of turbine design in America, probably no factor has played a more important part than the Holyoke testing flume. Up to the present time, more than 1 800 turbines have been tested there, and these tests have undoubtedly had a very important influence on the advancement of the art. The fact that all wheels (within certain limits of capacity)* are tested there under essentially equivalent

* The Holyoke flume is best adapted to the testing of wheels from 27 to 42 in. in diameter, approximately. At the time the flume was built, wheels were of much lower capacity, and it was not designed to handle the large volume of water discharged by high-speed wheels 45 in. in diameter or greater. The writer names 42 in. as the limit, not because he believes that it is impossible to get good results with a wheel of larger size, but because no engineer of judgment would think of installing a larger wheel to operate under similar conditions, and dependence cannot be placed on such tests. If the conditions are less favorable than would be found in an ordinary plant, they are certainly unsuitable for a test where the object is to determine the best performance of which the wheel is capable. The main defect in design which affects the tests of large wheels is the shallowness of the tail-race. The floor of the flume, on which the draft-tube of the wheel stands for test, is only 5 ft. above the flat bottom of the tail-race. The draft-tube of a 49 or 50-in. wheel, unless it is very short, will be about 7 ft. in diameter at the lower end. The back pressure due to a column of water 7 ft. in diameter impinging on the flat bottom of the tail-race, 5 ft. from the end of the draft-tube, can be easily imagined. It has a throttling effect upon the discharge which cuts down the effective head, but does not show on the gauge, and hence the efficiency, as shown by the test, suffers. The loss does not seem to be altogether proportionate to the quantity discharged, but, apparently, is affected very largely by the relation of the draft-tube diameter to the depth of the tail-race in which the column has to turn to flow toward the weir. A 7-ft. column of water cannot readily be turned through an angle of 90° in a space of 5 ft. without considerable disturbance. The writer on one occasion tested a wheel which discharged 275 cu. ft. per sec. and found that the tail water level, as determined by pressure readings from the draft-tube, was actually 1 ft. higher than the level in the well where the head gauge floats. The head, as shown by the gauge, was about 10 ft., whereas the real head on the wheel was only about 9 ft. Naturally it is useless to expect good efficiency under such conditions. This is an extreme case, but wheels of smaller size, down to about 42 in. in diameter, undoubtedly suffer in the same way, although to a less extent.

conditions, adds greatly to the value of the results, because it renders them directly comparable. Tests made in place are subject to the effects of so many local conditions that one scarcely knows when it is fair to make direct comparisons, but Holyoke tests are not subject to this uncertainty, because the conditions are known and constant.

Further than this, the writer believes that it is not going too far to state that the Holyoke tests are more accurate than a majority of those made elsewhere, and are as accurate as the best of them. There is no other device for measuring power which is as generally reliable as the friction brake, when accurately adjusted. At the Holyoke flume the power of all wheels is determined by friction brakes, of which there are several sizes. The water is measured, according to the Francis formula, by a sharp-crested weir of adjustable width. The width of the weir is adjusted according to the discharge of the wheel, in order to bring the head on the weir within the limits of the experiments on which the Francis formula is based. This is in accordance with the best practice elsewhere. Furthermore, the arrangements for taking simultaneous readings, and the system of testing in general, are much superior to the conditions which often obtain in the case of tests in place.

These facts, together with the unquestionable impartiality of the tests, have given them a position of generally accepted reliability. Certain imputations to the contrary have occasionally emanated from Europe, but these are best answered by the fact that, in not a few cases, European engineers who have transplanted their activities to America have failed conspicuously to attain in the Holyoke flume the good results with which they were accredited at home.

Holyoke Tests versus Tests in Operation.—It should not be thought, from the foregoing remarks, that the writer regards a Holyoke test as necessarily an exact determination of the duty which a turbine will perform after installation. It is scarcely ever that, although, by the exercise of proper judgment, it may be a reasonably reliable indication of what may be expected in regular operation.

Tests are of interest to the Engineering Profession at large for two reasons: first, in order to determine what results have been accomplished by a machine which has been built and installed; second, as evidence of what a builder has accomplished in the past, on which to base speculations as to what he can accomplish in the case of a

proposed installation. In the first case, a test in place is generally regarded as the most satisfactory, but, in the second case, the writer finds a great diversity of opinion regarding the relative merits of Holyoke tests and tests of units in regular operation. It is conceded by all, of course, that an accurate test in operation is the only exact method of determining the performance of the turbine under working conditions, and it is further conceded by most engineers that the tests which they themselves make are accurate. However, they are not so credulous in regard to the tests made by others. The apparatus used for tests at the site is usually improvised for the occasion, is handled, to a certain extent, by unpracticed hands, and the opportunities for error are many. For this reason some engineers place more confidence in Holyoke tests because they know that they are correct and that they furnish a reliable basis on which to estimate the results which may be expected after installation.

On the other hand, some assert that wheels are tested at Holyoke under ideal conditions which cannot be obtained in practice; that the test shows merely the efficiency of the runner, and a lot of other generalities, none of which has in general any sound foundation of fact. There are certain conditions under which a Holyoke test is more favorable to the wheel than a test in place, and there are other conditions under which it is decidedly the opposite. As a matter of fact, the results of any test, whether made at Holyoke or at the site, cannot be relied on to indicate the performance of a prospective installation under different conditions of head, or speed, or capacity, without very careful study of the influence which these changed conditions will have on the efficiency of the wheel. For example, to infer that, because a manufacturer has obtained certain efficiencies with a wheel developing 10 000 h.p. under 150 ft. head at 200 rev. per min., he can obtain the same results with a wheel developing 10 000 h.p. under 100 ft. head at 300 rev. per min. would be entirely unwarranted and erroneous. Such comparisons, however, are often made, sometimes to the disadvantage of the builder, and frequently to the delusion of the purchaser.

In order to elucidate this matter, the factors which affect a comparison between wheels operating under different conditions will be analyzed. For this purpose the losses of energy in the turbine may be divided as follows:

(*a*)—Hydraulic loss in the casing, from the penstock flange to the entrance to the guides,

(*b*)—Hydraulic loss in the guides and runner,

(*c*)—Hydraulic loss due to leakage around the runner,

(*d*)—Hydraulic loss in the draft-tube,

(*e*)—Mechanical loss due to the friction of the revolving parts.

In respect to loss (*a*), the advantage is usually in favor of the Holyoke test. It is ordinarily impracticable to test a runner at Holyoke in the casing in which it is to be permanently installed, and this fact must be taken into consideration. As installed in the flume, the wheel draws the water to the guide openings smoothly and with minimum disturbance, from a state of comparative rest. Naturally, the efficiency is higher under those conditions than it will be when the wheel is installed in a usually all too constricted casing in which the water approaches the guides with whirls and eddies due to obstructions and restricted areas. There is no form of casing for a turbine that can be made within the limits of reasonable proportions, except the spiral, which does not cause considerable loss of efficiency. The loss in a properly designed spiral casing is relatively small, because it accelerates the velocity gradually and delivers the water to the guides without abrupt change of direction. A test of a wheel in an open flume can be fairly applied to a turbine of the spiral-casing type with very little allowance for additional loss, but a considerable deduction should be made if the wheel is to be installed in a cylindrical casing.

Loss (*a*) is dependent largely on the velocity in the casing, and is related to the head only in that for a given velocity the percentage of loss decreases as the head increases.

Loss (*b*) is independent of the head, provided the speed is allowed to vary as $\sqrt{h}$, but is materially affected by the type of runner and the design of the guides. The results obtained with one type of runner cannot be assumed to apply to another type. This point is fully treated in the paper.

Loss (*c*) is likewise independent of the head, and is as great at Holyoke as in any other case. Both the leakage around the runner and the discharge through it vary as the $\sqrt{h}$, and hence the ratio of one to the other (or the percentage of loss) does not vary at all.

Loss (*d*), in the case of small wheels, is probably as low at Holyoke as elsewhere. With large wheels, however, this loss is excessive.*

Loss (*e*) varies with the head, decreasing as the latter increases, because the power of the wheel increases very much faster than the friction loss due to the weight of the revolving parts. There seems to be a general impression abroad that in testing wheels at Holyoke the mechanical friction is so far minimized as to be practically eliminated. It is true that effort is made to reduce the friction load, but it is not eliminated, by any means, and, although the percentage of loss is less than it would be for the same wheel running in horizontal bearings under the same head, it is more, and sometimes a great deal more, than it would be for the same wheel operating in a horizontal position under a higher head. In fact, it is necessary to minimize the friction in order to show results which will be representative of what the wheel can accomplish under heads of from 30 to 50 or 60 ft. For wheels to operate under higher heads, a Holyoke test is, almost without exception, a disadvantage to the wheel. This fact can be readily demonstrated. Consider, for example, one of the wheels referred to later, Test No. 1799. This wheel developed 112 h.p. under 17.25 ft. head. Suppose it is desired to install a pair of these wheels on a horizontal shaft under 80 ft. head. Will the efficiency be more or less than it was in the testing flume? Under this heading, of course, the influence of mechanical friction only will be considered.

This wheel when tested had four vertical guide-bearings, one ball thrust-bearing, and a stuffing-box, in contact with the shaft. The weight on the ball-bearing, including the unbalanced thrust of the runner, was about 7 000 lb. The pressure on the upper guide-bearing under the brake was 1 000 lb., due to the pull of the load. Under 80 ft. head, two of these wheels would develop

$$\frac{112 \times 2 \times \sqrt{80^3}}{\sqrt{17.25^3}} = 2\,240 \text{ h.p.},$$

which is twenty times as much power as was developed under test. The weight of the revolving parts would be about 5 000 lb.

Now, making the impossibly liberal assumption that under test the friction of the three lower guide-bearings, the stuffing-box, and the ball-bearing was absolutely nothing, there still remains the upper guide-

* See foot-note on p. 724.

bearing, which is known, beyond question, to carry a load of 1 000 lb. Since the bearings of the proposed unit will carry a weight of 5 000 lb., the friction will be about five times as great (neglecting the influence of speed, which would modify this statement slightly).

The power developed, however, is twenty times as great, and hence the percentage of energy lost is only one-quarter as much as when the wheel was tested at Holyoke. If the friction under test was 2%, then the friction in operation will be only 0.5%, or a difference of 1.5% in favor of the latter. Under yet higher heads, the wheel in operation has a continually increasing advantage from this source.

In spite of these facts, the writer has known engineers of experience to make the broad statement that friction is so completely eliminated from Holyoke tests that they may be considered to represent purely ideal results which cannot be obtained in practice; and this in reference to heads of even 200 or 300 ft. Such statements will not stand under analysis. Under high heads, exactly the reverse is true. In fact, the writer knows of no case so ideal or so favorable to high efficiency as that of a large reaction unit operating under a high head. In such a case, not only is the mechanical friction reduced to a minimum effect, but the casing usually takes the spiral form, which minimizes the casing loss. The wheel usually has individual draft-tubes, which minimize the draft-tube loss, and the design of the runner approaches the purely inward-flow Francis type, the characteristic of which is high efficiency. The design of the wheel throughout is the most efficient which can be devised.

Scope of Tests.—Tests of turbines after installation are usually made for the purpose of determining whether a specific set of conditions has been complied with satisfactorily. These tests are usually limited to observations at various gate openings under the head and speed for which the turbine was designed. Where the power is measured electrically, the difficulty of determining the efficiency of the generator at varying speeds precludes the possibility of a comprehensive test. Such tests may fulfil their function, which is to determine the performance of a guaranty, but they add very little to the knowledge of turbine characteristics in general.

It is customary practice to design a turbine to perform a certain duty under a certain head, and either to test it under that head, or else under whatever head is available, and, from the data thus

obtained, compute the performance of the wheel under the head for which it was designed. Excepting high heads, there are few installations, however, where the conditions are by any means constant. A test made at constant speed under one head, is of little value in determining the performance of the wheel at the same speed under another head. It is of even less value in determining the general characteristics of the wheel, and, as will be demonstrated later, these characteristics are of vital importance in considering its adaptability to a given service. The characteristic curves of a wheel cannot be plotted from a test at constant speed. It is essential that the speed should vary through a considerable range. In this respect, the Holyoke tests are excellent, and, but for the fact that they are so seldom published, they would furnish a vast amount of useful information.

Selection of Wheels.—The writer is of the opinion that more trouble has resulted from the misapplication of turbines to conditions for which they are not suited, than from any other of the many mistakes which have been made in the equipment of water-power plants. There are numerous instances all over the country where a wheel which would give good results if run at the proper speed, is being run at some other speed and giving correspondingly poor results. The cause of these mistakes is not far to seek. It lies chiefly in the failure to analyze the tests of these wheels and thus determine whether they are properly adapted to the existing conditions.

Comparatively few engineers realize to what an extent the performances of different types of turbines differ. The divergence is wide, and the speed selected determines largely the type of runner which must be used. In spite of this, the speed is usually determined from a consideration of generator economy alone, without regard to its effect on the economy of the turbine. Such a course is short-sighted and ill-considered, to say the least. It results in high-speed wheels being adopted where lower-speed wheels are more suitable for many reasons. Frequently, much care and ingenuity are devoted to shackling the manufacturer with rigid guaranties for apparatus which should never be used at all for the purpose intended. Premiums are even paid to secure results which, economically considered, would be expensive if obtained gratuitously.

There is a more and more insistent demand for high-speed wheels, without a proper consideration of the limited conditions to which they

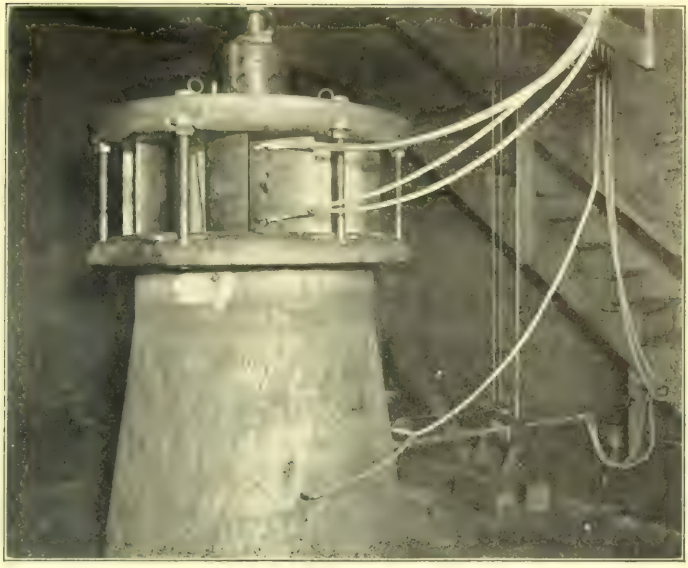


FIG. 1. WHEEL IN FLUME READY FOR A TEST.

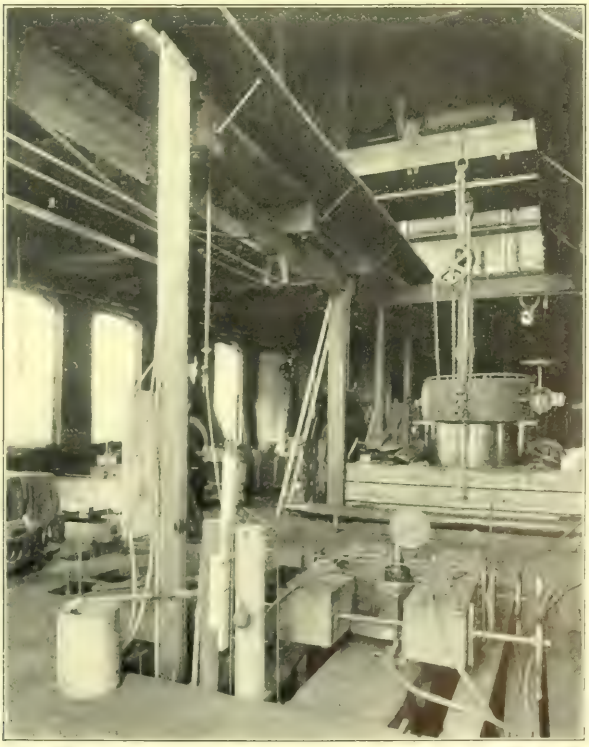


FIG. 2.—OPERATING FLOOR, SHOWING BRAKE, GAUGE-BOARD, ETC.

are adapted and the undesirable attributes which all of them possess. The high-speed wheel, as will be explained later, has its legitimate field, but in many plants where it is being used to-day it is by no means the most economical type.

Nor has the manufacturer failed of his share of responsibility in the misuse of his wheels. Because of the long-established practice in America of standardizing runners and building lines of stock sizes, there has been too much effort directed toward adapting the plant to the wheels, rather than the wheels to the plant. This unfortunate practice has been due largely to the "cut and try" methods of the McCormick school of turbine designers, which, until recent years, dominated the turbine business in America. Each builder, having by tedious experimentation evolved his standard patterns, was accustomed to select from his stock the wheel which, according to his judgment, would most closely approximate the desired results. Frequently, the approximation was a very remote one, not more, probably, because of the rigidity of his standards than because of his meager knowledge of their characteristics. It is only within the last few years that manufacturers have learned to analyze the tests of their wheels intelligently, and to predict with accuracy what they will do under a variety of conditions. It is still more recently that designing of special wheels to suit individual requirements has come into general practice.

The writer believes that the special wheel is to be the rule in the future, rather than the exception. This should not be taken to mean that every new wheel will necessarily be an entirely new and original design. Such a procedure would be of too experimental a nature, particularly in the case of mixed-flow wheels, which are not as completely amenable to mathematical analysis as are their more simple prototypes, the Francis wheels. New designs will consist rather of modifications of and improvements on wheels which have been tested and found to be efficient, and, as such, will be more certain of success than if based wholly on theoretical considerations. The important point to be observed is that the wheel should be exactly suited to the conditions, in order that the best results may be obtained. There should be no approximation.

Description of Tests.—The purpose of this paper is not to present a study of the various problems which arise in the selection of water-wheels, but to illustrate, by means of tests, the individual charac-

teristics of several different types, and to indicate in a general way the conditions of service to which each is best adapted. The conclusions drawn from this study are based, not merely on a consideration of these tests alone, but also of all the other American and European tests which have been accessible.

The tests which have been used for illustration were made recently at the Holyoke testing flume on a series of five specially designed runners. They are reproduced from the original reports in Tables 1 to 5, inclusive. These runners range from the low-capacity Francis type wheel, No. 1795, adapted particularly to high heads, to the large-capacity high-speed wheels, Nos. 1796 and 1797, adapted particularly to low heads. The average maximum efficiency of the five wheels is 88.06 per cent. This is believed to represent the best results thus far attained in the practice of the art. Tests Nos. 1799 and 1800 are especially notable in that they show higher efficiencies than any other reliable published records in either American or European practice. Test No. 1799, showing an efficiency of 90.43% at three-quarters-gate, is probably very close to the maximum obtainable economy.

The five wheels were successively tested on a vertical shaft in the same casing. The gates were of the balanced-wicket type. All the runners were of cast iron, made in one piece from cores.

Fig. 1 is a sectional view of the arrangement for testing one of the larger wheels. The vertical thrust was carried on a ball-bearing placed on top of the crown-plate. This arrangement was necessitated by the variation in height of the runners. It was not adopted to minimize the mechanical friction, and it is not believed that the friction was any less in the case of these tests than is usual with wheels of the same size. The usual arrangement is to carry the thrust on lignum vitæ step-bearings, but the invariable practice of all manufacturers is to turn off the face of the block until a small button about 2 in. in diameter is left for bearing. While running on this button the friction is practically nothing, although it takes more of a pull to start than is required with a ball-bearing.

Fig. 2 shows the arrangement for wheel No. 1795, with an interior wooden draft-tube. This inner tube was not used with any of the other wheels.

Fig. 1, Plate LVI, is a photograph of the complete apparatus erected in the flume ready for a test. This view shows an arrangement of

TABLE 1. TESTS OF A 32-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1795.

Date, February 18th and 19th, 1909. Case No. 1794.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
69	1.000	1.024	17.23	4	112.50	34.45	46.84	69.59
67	1.000	1.009	17.50	3	152.67	34.22	55.80	82.36
68	1.000	1.008	17.57	2	163.00	34.25	57.31	83.98
66	1.000	1.005	17.46	3	164.00	34.05	56.90	84.40
65	1.000	0.992	17.47	3	172.67	33.60	55.92	84.00
64	1.000	0.979	17.48	3	181.33	33.17	54.53	82.93
63	1.000	0.965	17.46	4	189.25	32.67	52.53	81.21
62	1.000	0.946	17.47	4	213.00	32.06	49.27	77.57
61	1.000	0.688	17.59	4	251.50	23.39	34.91	74.81
60	1.000	0.631	17.86	4	298.00	21.62	.....	.....
59	0.889	0.933	17.50	3	113.67	31.63	45.23	72.04
58	0.889	0.934	17.52	3	135.67	31.70	50.84	80.72
57	0.889	0.931	17.54	3	146.33	31.60	52.80	84.01
56	0.889	0.925	17.53	3	153.67	31.40	53.32	85.41
55	0.889	0.922	17.44	4	156.50	31.21	52.85	85.62
54	0.889	0.916	17.45	3	160.33	31.00	52.66	85.84
53	0.889	0.909	17.46	3	164.00	30.79	52.35	85.87
52	0.889	0.903	17.47	4	168.00	30.59	52.07	85.92
51	0.889	0.898	17.48	4	172.75	30.42	51.95	86.14
50	0.889	0.893	17.48	4	176.75	30.25	51.52	85.91
49	0.889	0.886	17.49	4	184.00	30.05	51.08	85.69
48	0.889	0.864	17.53	4	206.50	29.31	47.77	81.98
47	0.889	0.795	17.59	4	244.75	27.02	33.97	63.02
46	0.889	0.572	17.91	4	298.75	19.61	.....	.....
45	0.741	0.809	17.57	4	106.00	27.48	39.23	71.65
44	0.741	0.801	17.57	4	139.50	27.20	45.18	83.35
43	0.741	0.791	17.55	3	152.00	26.85	45.71	85.53
41	0.741	0.784	17.57	4	159.75	26.63	45.82	86.35
40	0.741	0.780	17.60	4	166.25	26.52	46.15	87.18
42	0.741	0.777	17.56	4	171.50	26.40	46.02	87.53
39	0.741	0.770	17.64	4	180.75	26.22	45.99	87.68
38	0.741	0.758	17.67	4	191.50	25.82	44.30	85.61
37	0.741	0.727	17.73	3	210.67	24.81	38.99	78.15
36	0.741	0.693	17.79	3	231.33	23.68	32.11	67.20
35	0.741	0.503	18.03	3	298.00	17.31	.....	.....
34	0.593	0.669	17.84	3	101.00	22.89	32.71	70.63
33	0.593	0.660	17.84	3	122.67	22.61	36.32	79.40
31	0.593	0.644	17.86	3	145.00	22.07	37.57	84.04
30	0.593	0.642	17.85	3	151.33	21.98	37.81	84.97
32	0.593	0.637	17.88	4	158.00	21.83	38.01	85.87
29	0.593	0.632	17.91	4	163.75	21.68	37.88	86.02
28	0.593	0.622	17.91	3	173.00	21.33	36.82	84.98
27	0.593	0.606	17.99	3	188.00	20.82	34.79	81.90
26	0.593	0.537	18.16	4	239.00	18.54	22.11	57.92
25	0.593	0.496	18.05	3	281.00	13.98	.....	.....
24	0.444	0.505	17.91	3	94.00	17.32	23.92	67.99
22	0.444	0.489	17.92	2	130.00	16.78	27.06	79.36
21	0.444	0.486	17.94	3	136.00	16.70	27.08	81.48
23	0.444	0.486	17.92	4	139.25	16.66	27.70	81.82
20	0.444	0.481	17.94	3	145.33	16.50	27.57	82.12
19	0.444	0.472	17.94	4	153.00	16.21	26.90	81.53
18	0.444	0.465	17.95	5	161.40	15.98	26.13	80.34
17	0.444	0.450	17.99	4	178.00	15.46	24.71	78.32
16	0.444	0.402	18.04	4	231.50	13.85	16.07	56.70

TABLE 1.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
15	0.444	0.325	18.17	4	281.25	11.22	.....	.....
14	0.296	0.325	18.19	3	89.67	11.22	15.35	66.32
13	0.296	0.320	18.21	3	107.00	11.06	16.58	72.60
11	0.296	0.317	18.23	2	114.67	10.97	16.98	74.85
12	0.296	0.316	18.22	3	118.33	10.94	16.97	75.07
10	0.296	0.313	18.24	3	122.67	10.82	17.03	76.07
9	0.296	0.308	18.25	4	130.75	10.68	16.94	76.62
8	0.296	0.304	18.26	4	137.75	10.54	16.57	75.91
7	0.296	0.300	18.30	4	146.50	10.41	16.27	75.29
6	0.296	0.295	18.31	4	154.75	10.23	15.75	74.15
5	0.296	0.291	18.32	4	164.00	10.10	15.17	72.31
4	0.296	0.283	18.33	5	181.20	9.82	14.25	69.81
3	0.296	0.276	18.36	4	191.25	9.58	13.27	66.53
2	0.296	0.266	18.33	4	208.00	9.23	11.55	60.18
1	0.296	0.225	18.37	4	261.00	7.83	.....	.....

NOTE.—For Experiments Nos. 1, 15, 25, 35, 46 and 60, the jacket was loose.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 1 300 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 2.4 ft. from the center of the shaft, sufficed to start the wheel.

Pitot tubes for measuring the velocity at the top and bottom of the guides and in the draft-tube. The latter consisted of a movable tube, with velocity and pressure points, introduced through a gland in the side of the draft-tube about 2 ft. below the bottom of the runner. This tube was arranged to move horizontally across the diameter of the draft-tube and also to revolve about its own axis. By means of these two movements, the velocity at any distance from the center of the shaft could be determined, and also the angle of whirl. The latter would necessarily be the angle of the velocity point which gave the highest reading. Both movements of the tube were controlled from the floor above, and, by means of indicators, the exact position of the point was at all times known.

Fig. 2, Plate LVI, is a photograph of the operating floor, showing the brake for measuring the power of the wheel. At the left is the gauge-board and auxiliary apparatus connected to the Pitot tubes below. The water columns were drawn up into view by means of a vacuum. The arrangement of this apparatus is shown diagrammatically in Fig. 3. The position of the columns in the glass tubes was controlled

TABLE 2. Tests of a 28-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1796.

Date, February 25th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
95	1.077	0.971	17.11	4	153.00	97.00	125.20	66.52
94	1.077	1.017	16.97	3	199.67	101.16	147.66	75.84
93	1.077	1.036	16.94	3	224.33	102.98	156.38	79.04
92	1.077	1.053	16.89	3	239.33	104.50	159.58	79.72
91	1.077	1.061	16.87	3	247.33	105.22	161.17	80.06
89	1.077	1.068	16.81	3	253.67	105.70	161.45	80.12
90	1.077	1.072	16.80	3	259.00	106.08	161.71	80.01
88	1.077	1.079	16.82	4	267.50	106.82	162.15	79.58
87	1.077	1.026	17.05	3	294.67	102.27	125.03	63.23
86	1.000	0.913	17.28	3	147.00	91.65	120.29	66.98
85	1.000	0.957	17.16	2	190.50	95.70	144.34	77.50
84	1.000	0.972	17.14	3	211.67	97.10	152.69	80.89
83	1.000	0.981	17.13	3	225.00	98.03	156.85	82.36
82	1.000	0.990	17.11	3	232.67	98.90	157.96	82.31
80	1.000	0.996	17.09	4	240.25	99.43	160.19	83.13
81	1.000	1.003	17.07	3	247.33	100.07	161.17	83.19
79	1.000	1.004	17.07	4	252.25	100.14	160.55	82.82
78	1.000	1.001	17.13	4	259.00	100 00	157.00	80.82
77	1.000	0.983	17.22	3	268.33	98.43	146.39	76.15
76	1.000	0.911	17.47	4	293.50	91.96	106.75	58.59
106	0.923	0.868	17.52	4	143.25	87.24	115.49	67.39
105	0.923	0.899	17.24	5	176.00	90.12	135.49	76.90
104	0.923	0.920	17.15	4	201.00	91.96	146.21	81.74
101	0.923	0.931	16.93	5	213.20	92.52	148.62	83.66
100	0.923	0.936	16.93	6	220.67	92.96	150.48	84.31
99	0.923	0.942	16.91	4	227.25	93.51	151.52	84.50
102	0.923	0.945	16.93	4	232.00	93.82	151.88	84.31
103	0.923	0.945	17.04	4	235.50	94.14	152.74	83.96
98	0.923	0.945	16.93	4	237.75	93.82	151.32	84.00
97	0.923	0.924	17.02	4	254.50	92.04	138.84	78.15
96	0.923	0.823	17.25	4	288.25	82.47	87.36	54.15
42	0.923	0.870	17.17	3	146.33	87.02	116.20	68.57
41	0.923	0.895	17.10	4	170.00	89.37	130.87	75.51
40	0.923	0.921	17.04	4	202.75	91.80	146.25	82.44
39	0.923	0.928	16.99	3	208.67	92.35	147.99	83.17
35	0.923	0.932	17.03	4	216.25	92.81	150.75	84.10
38	0.923	0.937	16.97	4	220.00	93.20	151.36	84.38
36	0.923	0.939	17.01	4	223.75	93.44	152.58	84.65
34	0.923	0.940	17.02	4	226.25	93.66	150.86	83.45
37	0.923	0.944	16.97	3	238.33	93.90	151.69	83.94
33	0.923	0.921	17.13	4	256.25	92.05	139.80	78 18
32	0.923	0.823	17.27	3	288.00	82.54	87.29	53.99
31	0.923	0.730	17.50	4	334.75	73.75	.....	.....
74	0.846	0.824	17.46	3	158.67	83.15	129.23	73.02
75	0.846	0.836	17.46	3	175.67	84.35	129.91	77.78
72	0.846	0.861	17.34	5	202.20	86.50	143.40	84.30
70	0.846	0.865	17.33	4	209.00	86.95	145.69	85.25
71	0.846	0.868	17.33	3	215.00	87.24	147.27	85.89
73	0.846	0.870	17.34	4	219.25	87.47	148.19	86.15
69	0.846	0.869	17.32	4	221.25	87.32	147.53	86.01
68	0.846	0.866	17.33	4	227.75	87.02	144.96	84.76
67	0.846	0.858	17.36	4	231.75	86.25	140.48	82.73
66	0.846	0.845	17.39	4	243.75	85.11	132.98	79.22
65	0.846	0.828	17.44	4	256.50	83.44	124.39	75.37

TABLE 2.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
64	0.846	0.754	17.59	3	282.00	76.31	85.47	56.15
30	0.769	0.750	17.38	3	141.00	75.52	102.56	68.90
26	0.769	0.766	17.35	3	166.00	77.02	115.72	76.36
27	0.769	0.779	17.32	3	183.00	78.24	124.24	80.84
25	0.769	0.789	17.31	3	194.00	79.25	129.36	83.15
29	0.769	0.793	17.25	4	200.75	79.48	131.42	84.52
28	0.769	0.792	17.26	4	206.00	79.40	131.11	84.36
24	0.769	0.773	17.38	4	226.25	77.82	123.43	80.47
23	0.769	0.735	17.49	3	251.33	74.17	106.64	72.49
22	0.769	0.690	17.56	3	269.67	69.82	81.73	58.78
21	0.769	0.623	17.68	4	323.75	63.27	.....	.....
20	0.615	0.617	17.91	4	139.50	63.07	88.79	69.31
16	0.615	0.627	17.79	3	158.33	63.81	95.97	74.55
17	0.615	0.634	17.80	3	171.33	64.55	101.26	77.71
15	0.615	0.638	17.73	4	179.50	64.82	103.37	79.31
18	0.615	0.636	17.77	4	183.00	64.74	103.16	79.07
19	0.615	0.634	17.80	2	188.00	64.61	102.56	78.64
14	0.615	0.627	17.72	4	194.50	63.74	100.21	78.24
13	0.615	0.596	17.77	3	218.67	60.60	92.78	75.97
12	0.615	0.563	17.79	4	243.00	57.29	73.65	63.72
11	0.615	0.519	17.92	4	312.00	53.00	.....	.....
10	0.462	0.452	17.34	3	117.33	45.40	56.90	63.73
5	0.462	0.453	17.02	3	136.00	45.65	61.83	70.17
7	0.462	0.461	17.08	4	146.75	46.04	64.94	72.82
6	0.462	0.462	17.05	4	152.00	46.04	66.34	74.52
4	0.462	0.462	16.92	4	155.25	45.90	65.88	74.79
9	0.462	0.459	17.16	3	162.00	45.94	66.78	74.69
8	0.462	0.457	17.15	3	166.67	45.69	65.67	73.90
3	0.462	0.451	16.98	4	172.25	44.83	62.65	72.57
2	0.462	0.432	17.05	4	217.50	43.04	52.74	63.37
1	0.462	0.404	17.18	5	282.40	40.40	.....	.....

NOTE.—During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling, was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

by three-way cocks. By connecting a set of tubes with the vacuum tank, the columns were drawn above the floor level, after which, by closing the cock, they were held in a stationary position. If drawn too high they could be dropped back by opening the cock to the atmosphere. After the columns were once adjusted and the cocks closed, they would stand indefinitely without attention.

This apparatus for the measurement of velocities has been described at some length for the benefit of those who may be interested in the same line of experiments. Lack of space precludes the publication of any of the data obtained, although they are of great interest to the designer, and throw much light on some very obscure points in the theory of turbine design.

TABLE 3.—Tests of a 30-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1797.

Date, February 26th and 27th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
82	1.000	0.942	17.06	3	149.00	94.89	126.41	68.85
81	1.000	0.950	16.99	3	160.33	95.52	131.16	71.26
80	1.000	0.963	17.00	4	190.50	96.85	144.30	77.28
75	1.000	0.977	17.02	4	210.50	98.34	150.52	79.30
74	1.000	0.984	16.95	4	220.00	98.81	153.31	80.72
76	1.000	0.992	17.01	4	227.25	99.75	155.61	80.87
78	1.000	0.994	16.94	4	229.50	99.75	155.76	81.28
77	1.000	0.997	16.98	4	231.25	100.14	155.55	80.66
79	1.000	1.002	16.90	3	239.67	100.45	156.85	81.47
73	1.000	1.003	16.94	4	254.75	100.70	154.37	79.80
72	1.000	0.871	17.13	3	287.00	87.93	86.96	50.91
71	1.000	0.728	17.44	3	317.67	74.17	.....	.....
52	0.923	0.875	17.08	3	131.33	88.14	111.42	65.26
51	0.923	0.896	16.95	3	156.33	89.97	127.89	73.95
50	0.923	0.913	16.93	4	188.25	91.66	142.59	81.02
44	0.923	0.920	16.89	4	201.25	92.20	146.34	82.86
45	0.923	0.927	16.85	4	211.50	92.81	148.67	83.83
46	0.923	0.931	16.86	4	217.00	93.20	149.91	84.12
49	0.923	0.934	16.83	4	220.50	93.42	150.99	84.68
47	0.923	0.936	16.83	4	223.75	93.60	151.86	85.00
48	0.923	0.937	16.83	3	226.33	93.74	150.87	84.32
43	0.923	0.929	16.88	4	241.75	93.13	146.49	82.17
42	0.923	0.788	17.21	4	276.25	79.76	83.70	53.77
41	0.923	0.670	17.44	4	312.50	68.26	.....	.....
70	0.846	0.868	17.07	4	192.75	87.47	142.50	84.15
67	0.846	0.875	17.10	4	205.00	88.22	147.83	86.41
68	0.846	0.876	17.06	4	209.50	88.22	148.53	87.02
66	0.846	0.876	17.11	4	212.00	88.37	149.02	86.91
69	0.846	0.877	17.05	3	213.00	88.30	148.43	86.94
65	0.846	0.876	17.13	5	213.80	88.37	147.70	86.03
64	0.846	0.875	17.13	5	215.80	88.30	146.46	85.38
63	0.808	0.810	17.33	4	143.50	82.24	114.78	71.01
62	0.808	0.834	17.25	3	177.00	84.44	135.14	81.81
61	0.808	0.843	17.24	5	194.80	85.35	142.83	85.59
60	0.808	0.847	17.21	3	201.00	85.73	144.94	86.62
59	0.808	0.848	17.19	3	206.00	85.73	146.05	87.39
58	0.808	0.847	17.19	4	208.25	85.65	145.12	86.91
57	0.808	0.845	17.19	3	210.00	85.41	143.80	86.36
56	0.808	0.840	17.22	3	216.67	85.05	141.80	85.37
55	0.808	0.825	17.25	4	224.75	83.53	136.19	83.34
54	0.808	0.807	17.29	4	236.50	81.80	128.98	80.41
53	0.808	0.695	17.51	4	264.25	70.97	80.06	56.81
40	0.769	0.755	17.29	4	118.50	76.53	93.35	62.21
39	0.769	0.776	17.21	4	146.50	78.47	110.97	72.46
35	0.769	0.800	17.03	4	174.75	80.49	127.07	81.74
36	0.769	0.806	17.03	4	190.75	81.13	135.24	86.31
37	0.769	0.805	17.10	4	197.25	81.21	137.46	87.28
38	0.769	0.805	17.15	4	200.50	81.28	137.29	86.84
34	0.769	0.803	16.83	4	199.50	80.34	132.98	86.72
33	0.769	0.788	16.93	4	211.75	79.04	128.32	84.55
32	0.769	0.745	16.93	4	232.50	74.72	112.71	78.56
31	0.769	0.660	17.36	4	257.50	67.02	78.02	59.13
30	0.769	0.579	17.60	3	306.00	59.27	.....	.....
29	0.615	0.610	17.69	3	100.67	62.60	69.54	55.37

TABLE 3.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel, per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
28	0.615	0.621	17.64	3	127.67	63.60	85.10	66.89
25	0.615	0.643	17.60	3	167.33	65.76	104.44	79.57
27	0.615	0.644	17.57	4	175.50	65.83	107.41	81.89
24	0.615	0.643	17.61	4	178.00	65.76	107.86	82.13
26	0.615	0.641	17.59	3	179.67	65.55	106.70	81.60
23	0.615	0.482	17.96	3	296.67	49.84	.....	.....
20	0.615	0.634	17.88	4	155.00	65.37	101.44	76.53
21	0.615	0.634	17.80	5	151.40	65.22	98.17	74.56
22	0.615	0.639	17.69	3	160.33	65.50	102.98	78.37
19	0.615	0.644	18.08	5	174.20	66.80	111.89	81.69
18	0.615	0.643	18.11	3	180.00	66.73	112.35	81.97
17	0.615	0.638	18.07	4	183.50	66.19	111.20	81.98
16	0.615	0.636	18.03	4	185.00	65.90	109.86	81.53
15	0.615	0.630	17.96	3	190.00	65.15	109.38	82.43
14	0.615	0.609	17.81	4	201.00	62.66	103.53	81.80
13	0.615	0.574	17.79	4	216.75	59.07	91.94	77.15
12	0.615	0.553	17.54	5	245.20	56.46	74.29	66.15
11	0.615	0.483	17.71	5	294.60	49.58	.....	.....
8	0.462	0.475	17.82	3	104.33	48.95	58.16	58.80
7	0.462	0.483	17.75	4	139.75	49.58	71.98	72.12
6	0.462	0.482	17.76	3	147.00	49.51	73.93	74.14
5	0.462	0.481	17.80	4	156.75	49.45	75.99	76.12
9	0.462	0.477	17.93	2	162.00	49.22	76.57	76.51
10	0.462	0.472	17.97	3	167.00	48.78	75.90	76.35
4	0.462	0.464	17.88	5	173.60	47.80	73.64	75.97
3	0.462	0.436	18.05	4	216.50	45.13	65.60	71.01
2	0.462	0.415	18.08	4	246.25	43.02	44.77	50.75
1	0.462	0.381	18.27	4	280.50	39.75	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 23, 30, 41 and 71.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

Figs. 4 to 19, inclusive, are performance curves plotted from the test reports, on the common basis of 1 ft. head. The head at the testing flume fluctuates, and it is therefore necessary to reduce all readings to a common basis. With constant efficiency, the brake-horse-power varies as $\sqrt{h^3}$, and the speed and discharge as $\sqrt{h}$. This principle is true, theoretically, and has been repeatedly proven, experimentally, and all the computations have been made upon this basis.

Three sets of curves have been plotted for each of the five wheels. The first in order for each wheel is a set of brake-horse-power and efficiency curves for all gate openings plotted to speed. Each curve is designated by the gate opening which it represents. In addition to power and speed, the writer usually plots the discharge curves, but

TABLE 4.—Tests of a 31-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1799.

Date, March 2d and 3d, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse power developed by wheel.	Percentage of efficiency of wheel.
49	1.000	1.049	17.15	3	134.67	79.75	115.81	74.66
48	1.000	1.038	17.15	3	147.00	78.95	119.29	77.68
47	1.000	1.025	17.19	3	165.67	78.02	123.40	81.13
46	1.000	1.024	17.18	2	174.00	77.91	124.34	81.91
45	1.000	1.018	17.19	3	178.33	77.52	124.19	82.18
44	1.000	1.013	17.19	3	183.33	77.16	124.35	82.66
43	1.000	0.012	17.19	4	186.25	77.09	124.07	82.56
42	1.000	1.009	17.18	4	189.50	76.82	123.94	82.81
41	1.000	1.007	17.19	3	193.00	76.67	123.89	82.89
40	1.000	1.006	17.12	4	195.50	76.44	123.13	82.96
39	1.000	1.002	17.09	4	200.25	76.10	122.48	83.04
38	1.000	0.999	17.10	4	206.25	75.87	122.41	83.19
37	1.000	0.997	17.07	3	210.33	75.66	121.01	82.62
36	1.000	0.993	17.09	4	219.00	75.38	119.36	81.70
35	1.000	0.906	17.24	4	258.75	69.11	78.35	57.98
34	1.000	0.744	17.45	4	302.25	57.07	.....	.....
32	0.883	0.909	17.18	3	129.33	69.23	101.03	74.90
31	0.883	0.904	17.20	4	154.25	68.82	111.16	82.81
33	0.883	0.901	17.19	4	165.00	68.60	113.91	85.18
30	0.883	0.896	17.20	3	172.33	68.25	114.80	86.23
29	0.883	0.892	17.21	4	181.50	67.97	116.51	87.82
27	0.883	0.888	17.25	3	188.00	67.77	117.27	88.45
26	0.883	0.886	17.30	4	194.00	67.71	117.49	88.44
28	0.883	0.885	17.33	4	197.25	67.43	117.06	88.85
25	0.883	0.883	17.31	4	201.75	67.49	116.07	87.61
24	0.883	0.857	17.28	4	213.25	65.42	109.77	85.62
23	0.883	0.818	17.31	4	227.00	62.53	96.23	78.39
22	0.883	0.761	17.42	4	244.25	58.32	73.96	64.19
21	0.883	0.620	17.65	4	291.75	47.83	.....	.....
69	0.750	0.835	17.28	4	124.75	63.73	91.41	73.19
68	0.750	0.839	17.23	4	148.75	63.93	103.59	82.93
67	0.750	0.834	17.23	4	168.75	63.60	109.35	87.99
64	0.750	0.830	17.24	4	179.25	63.26	110.72	89.52
65	0.750	0.829	17.23	4	182.00	63.20	110.77	89.69
63	0.750	0.828	17.25	4	186.25	63.12	111.66	90.43
66	0.750	0.824	17.23	3	188.67	62.85	110.83	90 24
62	0.750	0.821	17.29	4	191.75	62.72	110.32	89.70
61	0.750	0.809	17.30	4	197.50	61.81	107.64	88.76
60	0.750	0.796	17.34	4	203.25	60.86	104.62	87.42
59	0.750	0.768	17.38	4	212.50	58.80	96.52	83.28
58	0.750	0.693	17.55	4	237.00	53.31	71.76	67.64
57	0.750	0.573	17.75	4	287.00	44.32	.....	.....
20	0.667	0.726	17.27	4	106.25	55.43	70.78	65.20
19	0.667	0.737	17.26	3	148.67	56.25	90.93	82.59
18	0.667	0.739	17.25	4	161.00	56.39	95.55	86.62
17	0.667	0.738	17.24	4	168.50	56.31	96.94	88.05
16	0.667	0.733	17.27	3	173.67	55.92	96.76	88.35
15	0.667	0.722	17.30	4	179.25	55.17	95.52	88.25
14	0.667	0.699	17.34	4	189.50	53.48	91.81	87.30
13	0.667	0.671	17.39	4	201.75	51.37	85.52	84.42
11	0.667	0.508	17.63	4	280.75	39.17	.....	.....
8	0.500	0.554	17.82	3	117.00	42.95	60.23	69.39
7	0.500	0.546	17.91	3	135.00	42.46	65.40	75.84
10	0.500	0.548	17.67	3	151.00	42.28	68.58	80.95

TABLE 4.—(*Continued.*)

Number of experiment.	Proportional part of: Percentage of full opening of speed-gate.	Proportional part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiments, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
6	0.500	0.548	18.05	4	157.50	42.72	71.54	81.80
9	0.500	0.547	17.71	4	157.00	42.28	69.41	81.73
5	0.500	0.539	18.12	5	167.60	42.11	71.05	82.10
4	0.500	0.512	18.13	4	187.00	40.02	67.95	82.58
3	0.500	0.488	18.18	4	213.00	38.21	58.05	73.68
2	0.500	0.460	18.07	4	232.00	35.92	42.15	57.26
1	0.500	0.402	18.20	3	275.00	31.50	.....	.....
52	0.333	0.362	18.18	3	96.00	28.35	34.88	59.68
55	0.333	0.361	18.01	3	117.33	28.10	39.08	68.09
51	0.333	0.348	18.19	4	133.00	27.25	40.27	71.64
54	0.333	0.347	18.21	3	139.67	27.20	40.60	72.28
53	0.333	0.340	18.22	3	143.67	26.62	39.15	71.18
56	0.333	0.333	18.06	4	148.00	26.00	37.64	70.69
50	0.333	0.316	18.26	3	201.67	24.82	36.64	71.28

Note.—For Experiments Nos. 1, 11, 21, 34, and 57, the jacket was loose.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

they have been omitted from this paper in order to economize space.* When included, such a diagram might be termed in common parlance a "personality chart," because it embodies all the characteristics and peculiarities of the wheel. From it separate performance curves can be plotted for any speed whatsoever within the range of the test. The heavy speed line drawn through each of these charts indicates the "normal speed" of the wheel, that is, the speed at which it gives its maximum efficiency.

The second set of curves shows the performance of the wheels along these heavy lines, or at normal speed. The gate openings are also plotted to discharge, so that it is possible to read the efficiency at any fractional part of the full gate opening as well as at any fractional part of the full discharge of the wheel.

The first set of curves shows, in addition to the information already referred to, a scale of values of ϕ, in the equation,

$$W_1 = \phi \sqrt{2\ gh},$$

where W_1 is the peripheral speed of the wheel, based on its rated

* For full details of this method of plotting test curves, see Chapter XVI of "Water Power Engineering," by Daniel W. Mead, M. Am. Soc. C. E.

TABLE 5.—Tests of a 31-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1800.

Date, March 4th and 5th, 1909.

Wheel supported on ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
65	1.000	1.052	17.88	3	112.33	64.87	82.99	64.91
64	1.000	1.040	17.41	3	133.33	64.20	91.24	71.98
63	1.000	1.025	17.43	4	154.25	63.33	96.21	76.85
62	1.000	1.014	17.42	3	169.67	62.60	98.64	79.76
61	1.000	1.009	17.40	3	183.33	62.25	99.92	81.34
60	1.000	1.006	17.42	4	193.25	62.13	100.65	82.00
59	1.000	1.004	17.43	4	201.00	62.01	101.03	82.42
58	1.000	1.002	17.44	4	207.00	61.88	101.54	82.96
56	1.000	1.000	17.35	4	211.00	61.65	100.95	83.22
57	1.000	0.999	17.39	4	216.75	61.65	101.07	83.13
55	1.000	0.998	17.36	4	221.50	61.53	100.60	83.05
54	1.000	0.995	17.38	4	247.50	61.40	97.42	80.50
53	1.000	0.951	17.45	4	266.75	58.80	80.77	69.41
52	1.000	0.730	17.71	4	321.25	45.44	.....	.....
51	0.883	0.889	17.48	4	132.25	55.02	79.29	72.69
50	0.883	0.888	17.48	4	147.00	54.91	83.68	76.88
49	0.883	0.885	17.47	4	165.00	54.71	88.93	82.04
48	0.883	0.879	17.39	4	179.25	54.24	91.18	85.24
47	0.883	0.876	17.42	5	189.20	54.11	91.66	85.75
44	0.883	0.873	17.54	4	197.50	54.11	93.29	86.67
43	0.883	0.873	17.40	4	204.25	53.85	92.77	87.30
45	0.883	0.872	17.40	3	209.00	53.79	92.40	87.05
46	0.883	0.871	17.42	4	215.50	53.79	92.66	87.19
42	0.883	0.858	17.40	3	225.00	52.92	88.57	84.81
41	0.883	0.810	17.45	3	248.00	50.06	75.09	75.80
40	0.883	0.609	17.68	3	314.00	37.87	.....	.....
75	0.733	0.801	17.76	4	158.50	49.93	81.59	81.13
73	0.733	0.798	17.73	3	183.33	49.70	86.60	86.66
72	0.733	0.797	17.71	3	190.33	49.63	87.60	87.88
71	0.733	0.796	17.72	3	197.00	49.57	88.28	88.62
74	0.733	0.795	17.73	3	199.00	49.50	87.97	88.39
70	0.733	0.793	17.72	3	201.67	49.37	87.93	88.63
69	0.733	0.788	17.70	4	204.50	49.02	86.69	88.10
68	0.733	0.775	17.71	4	213.50	48.27	84.04	86.69
67	0.733	0.722	17.79	3	234.00	45.04	70.85	77.97
66	0.733	0.544	18.03	3	308.67	34.15	.....	.....
38	0.667	0.751	17.58	3	171.00	46.58	78.70	84.75
37	0.667	0.750	17.57	4	184.00	46.52	81.34	87.75
39	0.667	0.749	17.56	4	187.00	46.45	81.54	88.14
35	0.667	0.749	17.59	4	185.50	46.45	80.88	87.29
36	0.667	0.747	17.59	4	188.75	46.38	81.73	88.33
34	0.667	0.742	17.58	4	192.50	46.02	80.44	87.67
33	0.667	0.731	17.59	4	198.25	45.37	78.04	86.32
32	0.667	0.753	17.70	3	127.67	46.87	68.04	72.32
31	0.667	0.752	17.68	3	144.67	46.76	72.72	77.56
29	0.667	0.752	17.66	4	164.25	46.76	77.59	82.85
30	0.667	0.753	17.68	3	178.67	46.82	81.15	86.44
28	0.667	0.750	17.62	3	189.00	46.57	82.11	88.56
27	0.667	0.741	17.62	3	193.67	46.02	80.93	88.00
26	0.667	0.731	17.64	3	201.00	45.44	79.12	87.04
25	0.667	0.725	17.63	3	204.00	45.05	77.83	86.41
24	0.667	0.709	17.64	3	213.33	44.05	74.93	85.03
23	0.667	0.678	17.70	4	224.50	42.23	67.98	80.19
22	0.667	0.506	17.85	3	303.33	31.61	.....	.....

TABLE 5.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
21	0.500	0.566	17.72	2	124.00	35.27	50.31	70.98
20	0.500	0.560	17.74	3	136.33	34.87	52.84	75.32
19	0.500	0.554	17.71	4	146.50	34.50	53.23	76.82
18	0.500	0.543	17.72	4	155.75	33.82	52.82	77.72
17	0.500	0.535	17.74	3	166.00	33.31	52.27	78.00
16	0.500	0.521	17.77	3	176.67	32.47	51.36	78.48
15	0.500	0.507	17.79	3	189.00	31.61	50.36	78.97
14	0.500	0.494	17.83	4	200.25	30.87	48.51	77.71
13	0.500	0.481	17.86	3	214.00	30.08	45.36	74.45
12	0.500	0.464	17.89	4	228.75	29.02	41.56	70.58
11	0.500	0.380	18.00	4	299.50	23.87	.....	.....
8	0.333	0.360	18.09	4	112.00	22.65	30.52	65.68
7	0.333	0.351	18.12	3	127.00	22.10	31.53	69.43
6	0.333	0.350	18.13	3	136.00	22.02	32.12	70.95
5	0.333	0.346	18.22	4	141.50	21.84	31.71	70.26
4	0.333	0.340	18.24	4	148.00	21.47	31.37	70.63
3	0.333	0.332	18.31	4	154.75	21.03	30.93	70.82
2	0.333	0.323	18.25	4	165.25	20.42	30.02	71.04
10	0.333	0.307	18.17	4	212.50	19.33	28.31	71.08
9	0.333	0.300	18.15	3	231.33	18.93	25.22	64.72
1	0.333	0.248	18.35	4	287.25	15.74	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 22, 40, 52, and 66.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain or 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

diameter. The third set of curves shows the performance of the wheels at speeds corresponding to several different values of ϕ. When a wheel operates at constant speed under a fluctuating head, the value of ϕ is continually changing. A comparison of these curves, therefore, is a very instructive study, because it shows how the efficiency varies as the head changes. The change in efficiency corresponding to a given change in ϕ is very different for different types of runner.

High-Speed Wheels.—In the use of the term "high-speed wheel," there seems to be much confusion. In the sense in which it is used here, the term does not apply necessarily to wheels of high peripheral speed, although a high value of ϕ is usually an attribute of a "high-speed wheel." As the term is used here, the wheel which, power for power, under a constant head, runs at the highest number of revolutions, is the highest-speed wheel. A high-speed wheel is one which will develop any given quantity of power at a high speed, relative to other wheels developing the same power under the same head. It is not

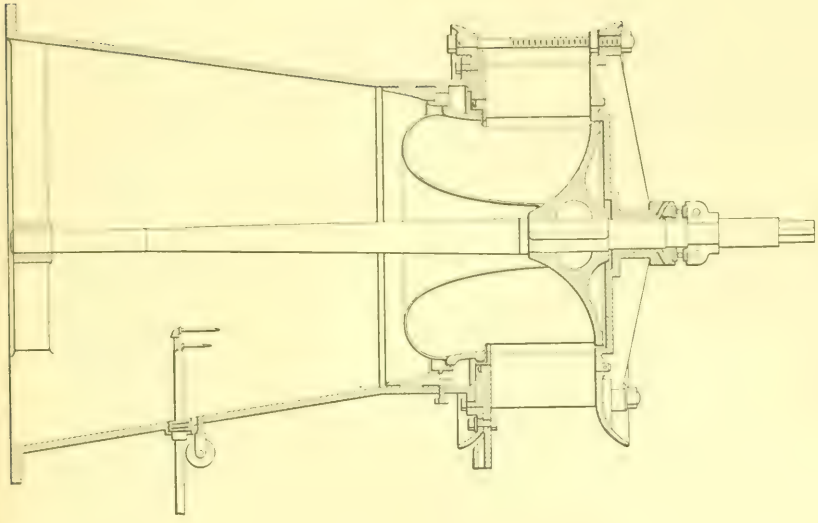


Fig. 1.

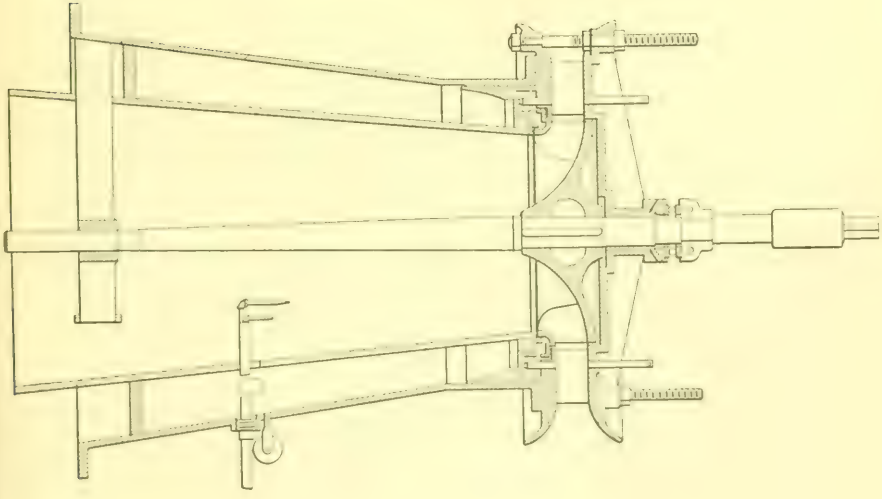


Fig. 2.

possible to compare the speeds of wheels upon the basis of diameter, because the power is not a consistent function of the diameter, except in wheels of homologous design. To determine which of two wheels is the higher-speed, it is usually necessary, unless they differ widely, to reduce them to a common basis of power and head.

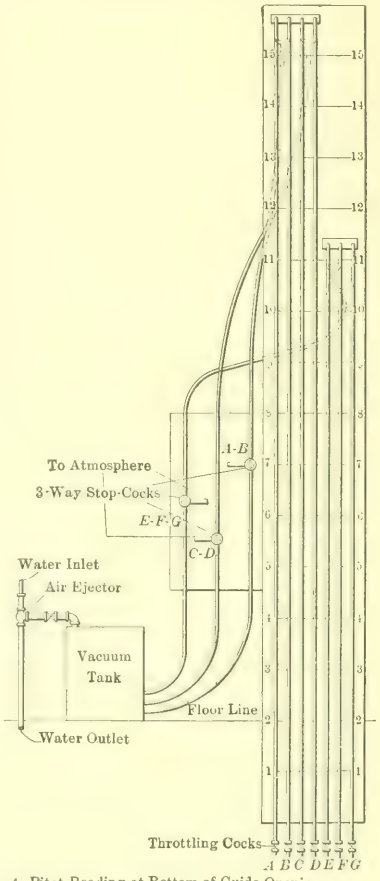
Suppose, for example, that two types of runner, *A* and *B*, are to be compared, and it is known from tests, or otherwise, that some certain size of *A* (the diameter of which need not be given) will develop a maximum of 2 080 h.p. at 500 rev. per min. under 100 ft. head, whereas an entirely different size of *B* will develop a maximum of 4 390 h.p. at 580 rev. per min. under 150 ft. head. It is impossible to determine from an inspection of these data which of these types is the higher-speed.

They should first be reduced to a common head, say 1 ft. Dividing the horse-power in each case by $\sqrt{h^3}$ and the speed by $\sqrt{h}$, the result is

A. .2.07 h.p. at 50 rev. per min.
B. .2.37 h.p. at 47.4 rev. per min.

A runs at a higher speed than *B*, but *B* develops more power than *A*. One cannot be sure but that if *B* were reduced in diameter until it developed the same power as *A*, it might run at as high a speed as *A*, or higher.

DIAGRAM OF GAUGE-BOARD AND AUXILIARY VACUUM APPARATUS FOR MEASURING VELOCITIES THROUGH GUIDE OPENINGS AND DRAFT-TUBE



A–Pitot Reading at Bottom of Guide Opening
B– Pressure " " " " "
C– Pitot " " Top " "
D– Pressure " " " " "
E– Pitot " from Movable Point in Draft-Tube
F– Pressure " " " " "
G–Pressure " " Wall of Draft-Tube at same Elevation as Movable Point.

FIG. 3.

Next reduce them to the same power, say 1 h.p. It is known that the power of wheels of homologous design varies directly as the

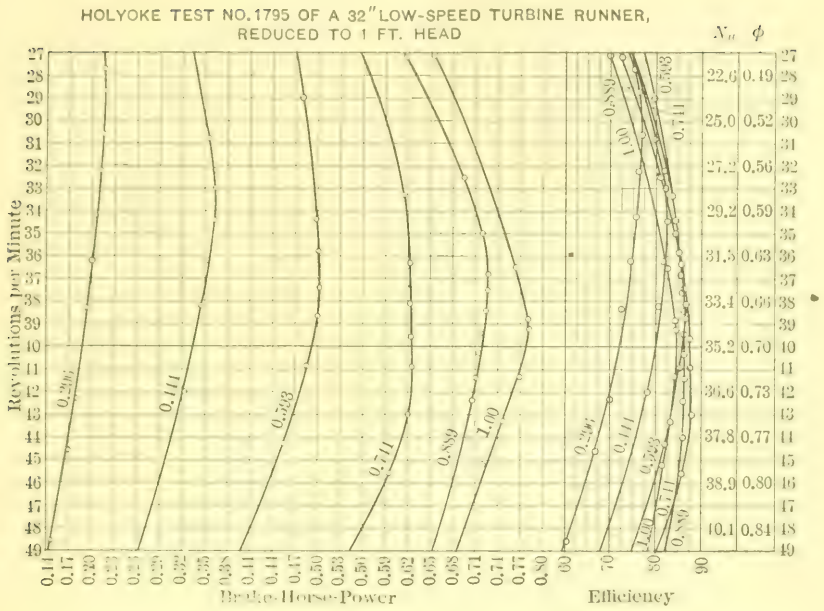

HOLYOKE TEST NO. 1795 OF A 32" LOW-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u ϕ
22.6 0.49
25.0 0.52
27.2 0.56
29.2 0.59
31.5 0.63
33.4 0.66
35.2 0.70
36.6 0.73
37.8 0.77
38.9 0.80
40.1 0.84
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.296
0.141
0.503
0.711
0.889
1.00
0.880
0.503
0.711
0.296
0.141
0.503
0.711
0.889
1.00

FIG. 4.

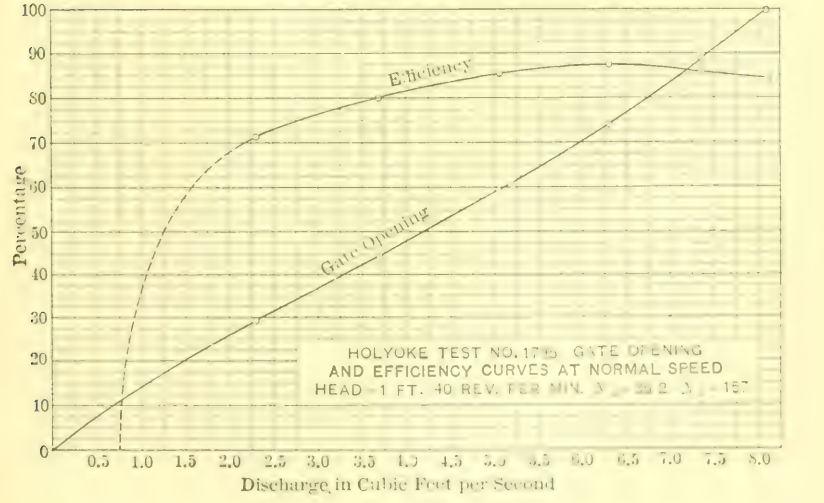

Efficiency
Gate Opening
Percentage
Discharge in Cubic Feet per Second
HOLYOKE TEST NO. 1795 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD 1 FT. 40 REV. PER MIN.

FIG. 5.

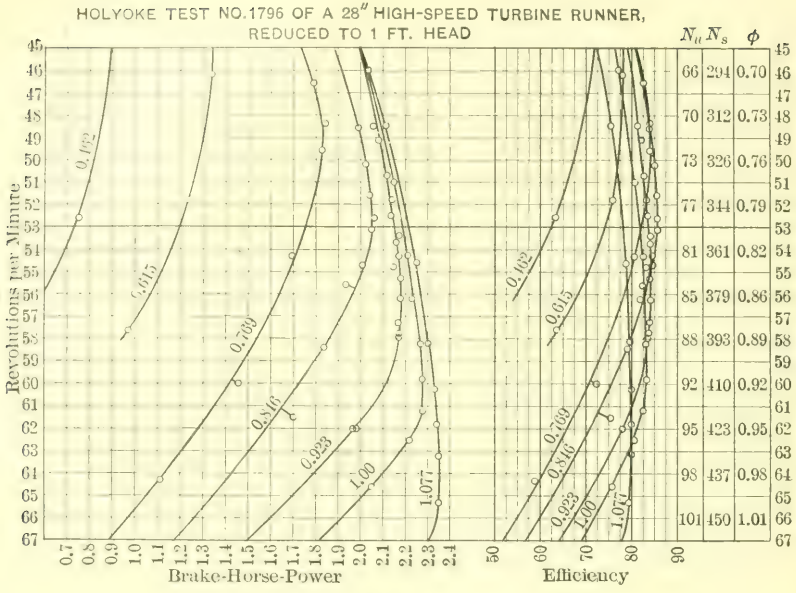

HOLYOKE TEST NO. 1796 OF A 28" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
66 294 0.70
70 312 0.73
73 326 0.76
77 344 0.79
81 361 0.82
85 379 0.86
88 393 0.89
92 410 0.92
95 423 0.95
98 437 0.98
101 450 1.01
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.102
0.615
0.769
0.846
0.923
1.00
1.077

FIG. 6.

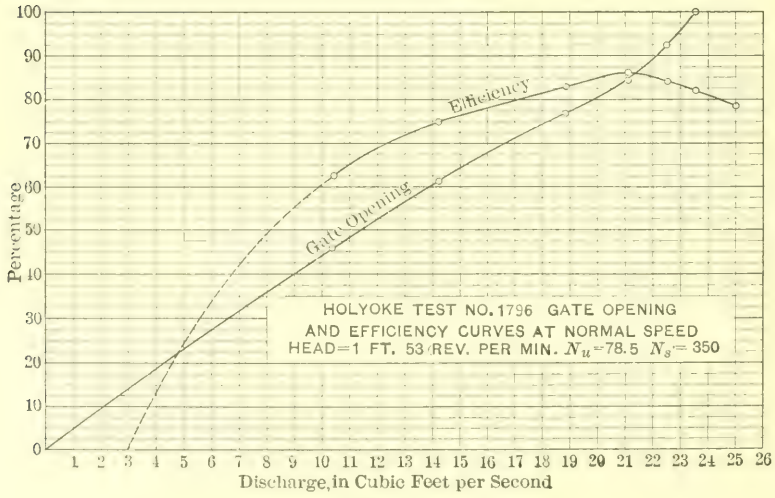

Percentage
Discharge, in Cubic Feet per Second
Efficiency
Gate Opening
HOLYOKE TEST NO. 1796 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 53 REV. PER MIN. N_u=78.5 N_s= 350

FIG. 7.

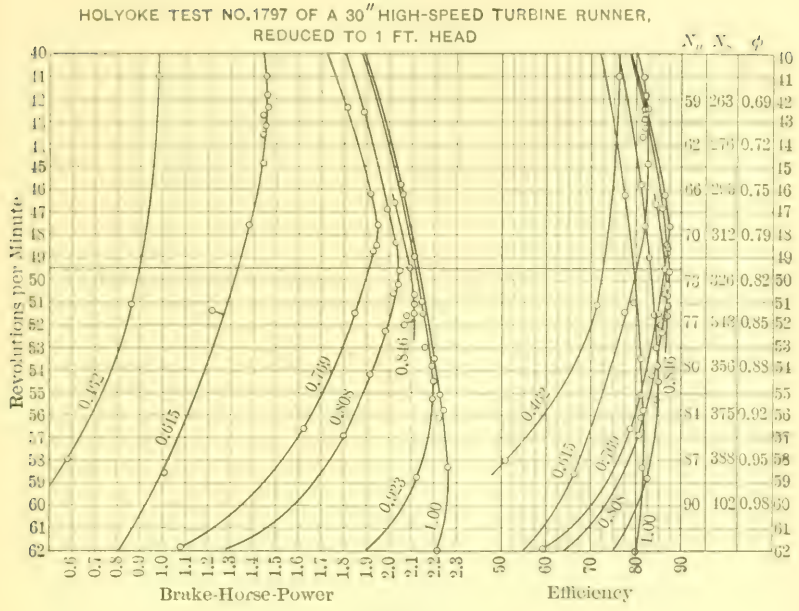

HOLYOKE TEST NO. 1797 OF A 30" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
59 263 0.69
62 276 0.72
66 293 0.75
70 312 0.79
73 326 0.82
77 343 0.85
80 356 0.88
84 375 0.92
87 388 0.95
90 102 0.98
0.427
0.615
0.799
0.908
0.846
0.923
1.00
Revolutions per Minute
Brake-Horse-Power
Efficiency

FIG. 8.

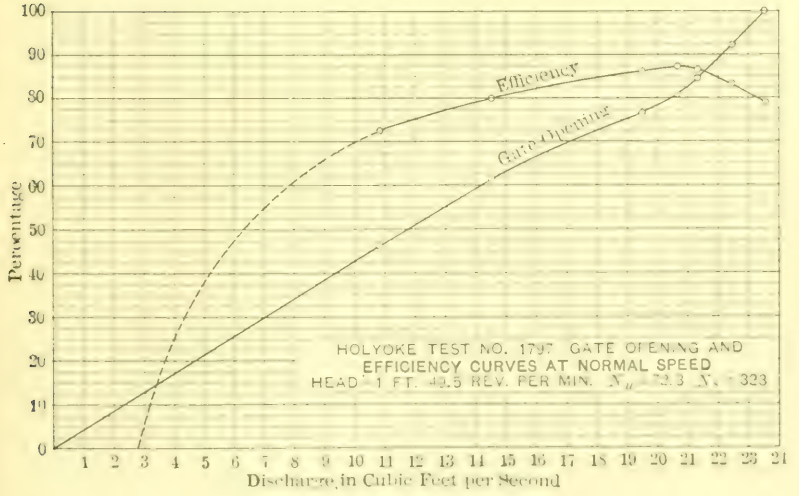

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO. 1797 GATE OPENING AND
EFFICIENCY CURVES AT NORMAL SPEED
HEAD = 1 FT. 49.5 REV. PER MIN. N_u = 72.3 N_s = 323
Discharge in Cubic Feet per Second

FIG. 9.

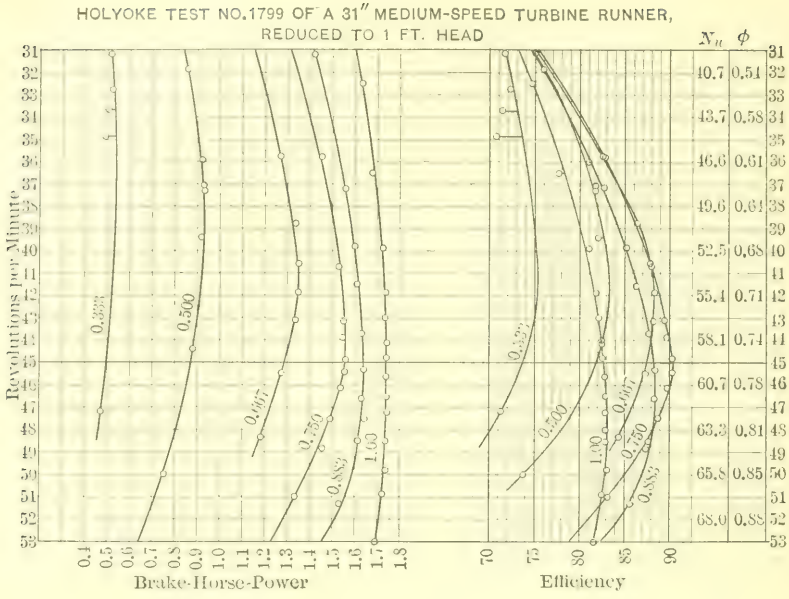

HOLYOKE TEST NO.1799 OF A 31″ MEDIUM-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u ϕ
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.333
0.500
0.667
0.750
0.833
1.00

FIG. 10.

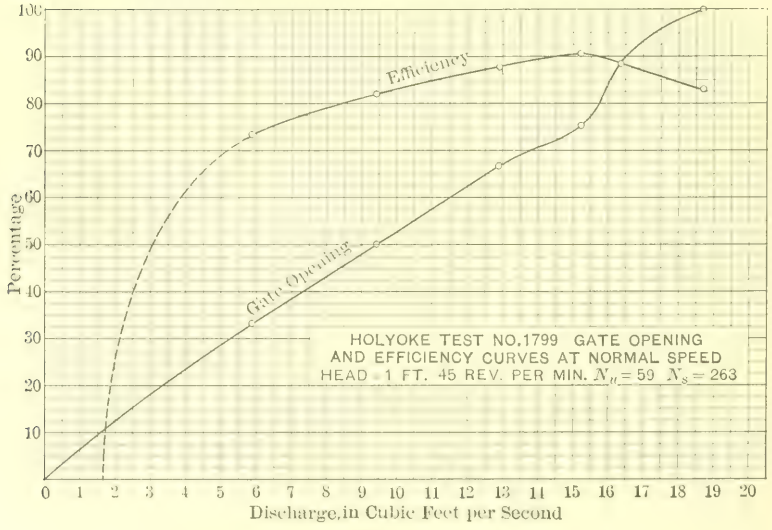

Efficiency
Gate Opening
Percentage
Discharge, in Cubic Feet per Second
HOLYOKE TEST NO.1799 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD = 1 FT. 45 REV. PER MIN. $N_u = 59$ $N_s = 263$

FIG. 11.

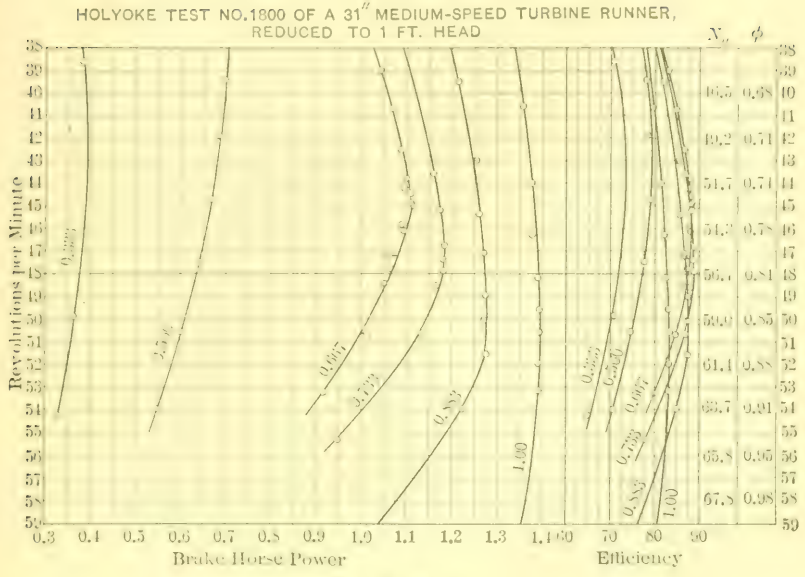

HOLYOKE TEST NO.1800 OF A 31" MEDIUM-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_1
ϕ
Revolutions per Minute
Brake Horse Power
Efficiency
0.333
2.5 "
0.667
3.333
0.833
1.00
0.333
0.500
0.667
0.833
1.00
46.5 0.68
49.2 0.71
51.7 0.74
54.2 0.78
56.1 0.81
59.0 0.85
61.4 0.88
63.7 0.91
65.8 0.95
67.8 0.98

Fig. 12.

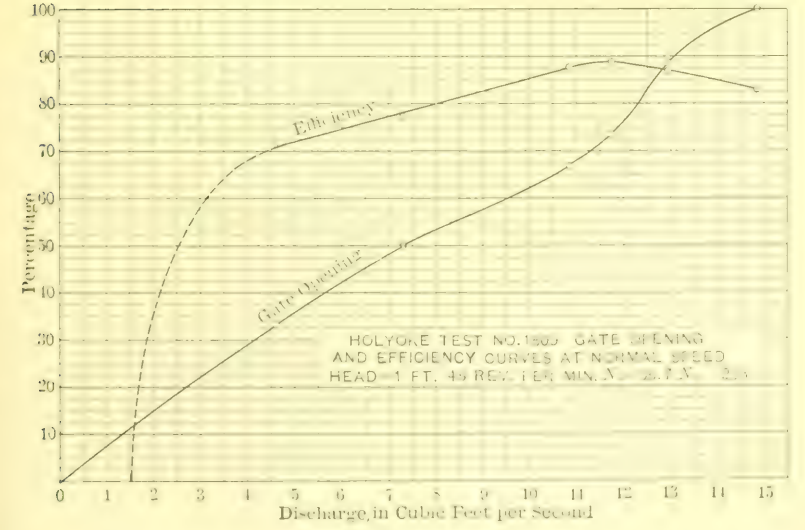

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO.1800 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD 1 FT. 45 REV. PER MIN. N_1 = 25 N_1 = 2.5
Discharge, in Cubic Feet per Second

Fig. 13.

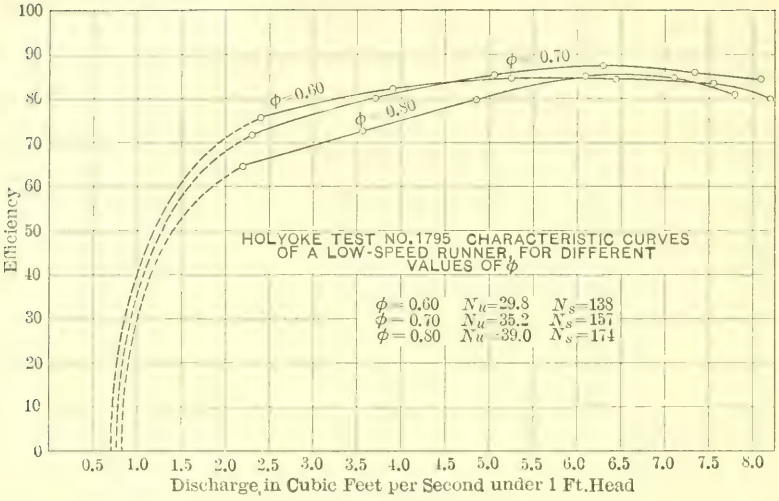

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1795 CHARACTERISTIC CURVES OF A LOW-SPEED RUNNER, FOR DIFFERENT VALUES OF φ
φ = 0.60 Nu = 29.8 Ns = 138
φ = 0.70 Nu = 35.2 Ns = 157
φ = 0.80 Nu = 39.0 Ns = 174
φ = 0.60
φ = 0.70
φ = 0.80

FIG. 14.

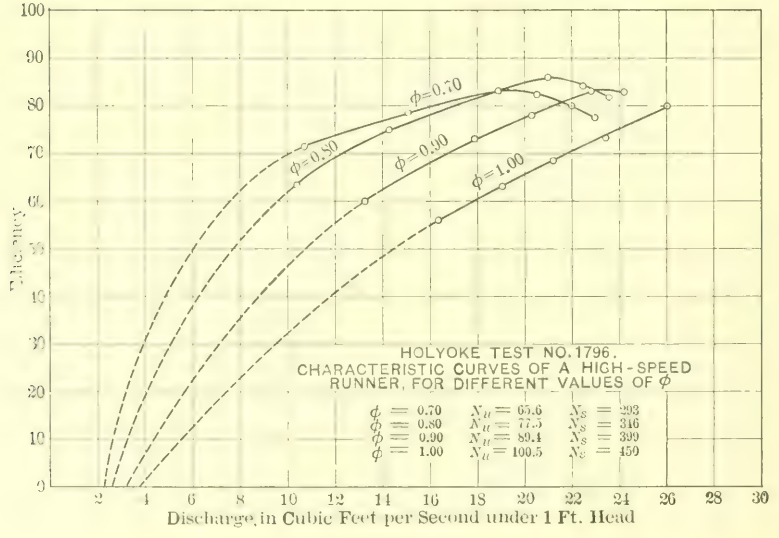

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1796. CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF φ
φ = 0.70 Nu = 65.6 Ns = 293
φ = 0.80 Nu = 77.5 Ns = 346
φ = 0.90 Nu = 89.4 Ns = 399
φ = 1.00 Nu = 100.5 Ns = 450
φ=0.70
φ=0.80
φ=0.90
φ=1.00

FIG. 15.

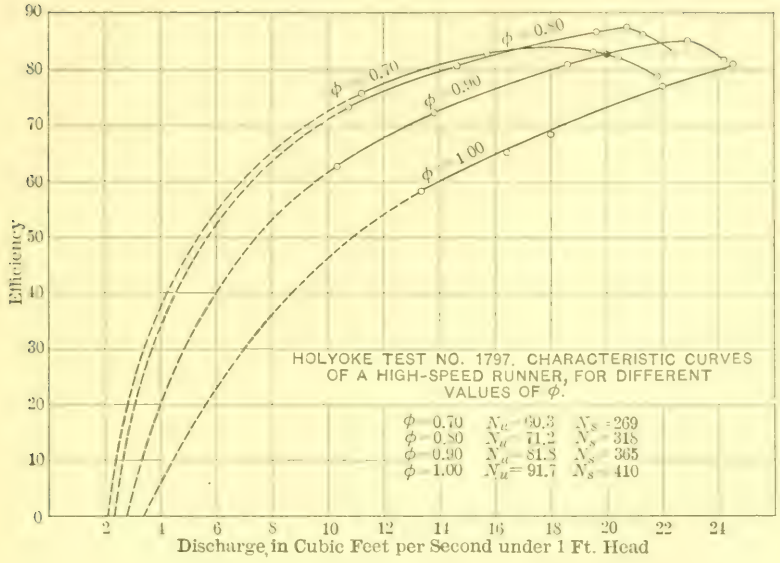

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1797. CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ.
ϕ 0.70 N_u 60.3 N_s 269
ϕ 0.80 N_u 71.2 N_s 318
ϕ 0.90 N_u 81.8 N_s 365
ϕ 1.00 N_u 91.7 N_s 410
ϕ 0.80
ϕ 0.70
ϕ 0.90
ϕ 1.00

FIG. 16.

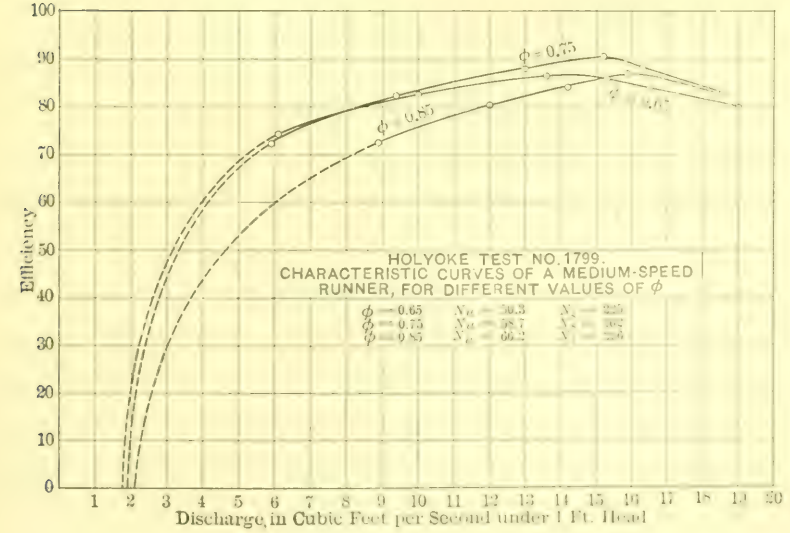

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1799. CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
ϕ 0.65 N_u 50.3 N_s 225
ϕ 0.75 N_u 58.7 N_s 305
ϕ 0.85 N_u 66.2 N_s 296
ϕ = 0.75
ϕ 0.85

FIG. 17.

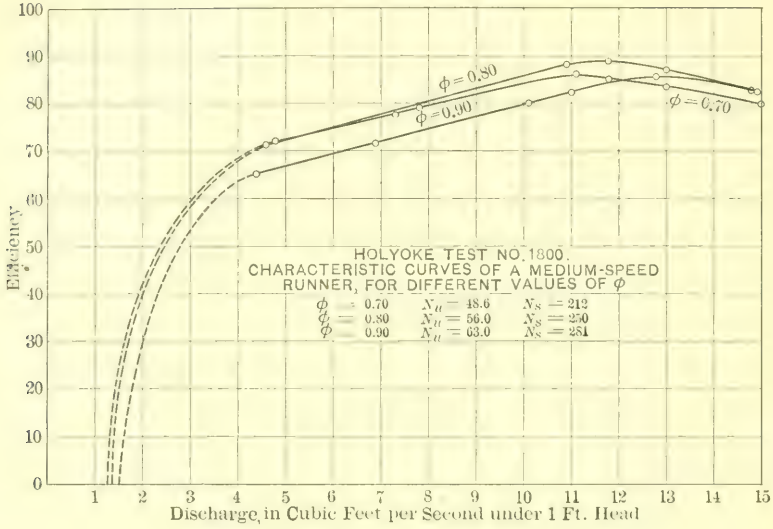

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.80$
$\phi = 0.90$
$\phi = 0.70$
HOLYOKE TEST NO. 1800.
CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.70$ $N_u = 48.6$ $N_S = 212$
$\phi = 0.80$ $N_u = 56.0$ $N_S = 250$
$\phi = 0.90$ $N_u = 63.0$ $N_S = 281$

FIG. 18.

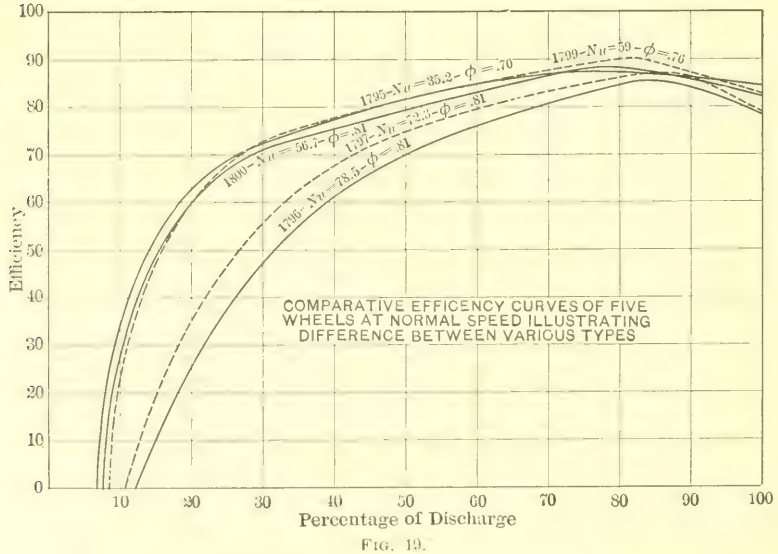

Efficiency
Percentage of Discharge
1795-N_u=35.2-ϕ=.70
1799-N_u=59-ϕ=.76
1800-N_u=56.7-ϕ=.81
1797-N_u=72.3-ϕ=.81
1796-N_u=78.5-ϕ=.81
COMPARATIVE EFFICENCY CURVES OF FIVE WHEELS AT NORMAL SPEED ILLUSTRATING DIFFERENCE BETWEEN VARIOUS TYPES

FIG. 19.

squares of the diameters, and that the speed varies inversely as the diameters, with constant efficiency.

Let D_a = the diameter of A,

D'_a = the diameter of a wheel of homologous design, having a capacity of 1 h.p.,

N'_a = the speed of the latter.

Then

$$D^2_a : D'^2_a = 2.07 : 1$$

$$D_a : D'_a = \sqrt{2.07} : 1$$

also

$$D_a : D'_a = N'_a : 50$$

then

$$\sqrt{2.07} : 1 = N'_a : 50$$

$$N'_a = 50\sqrt{2.07} = 72$$

Similarly $N'_b = 75$.

It is now plain that B is the higher-speed type, because, when both wheels are developing 1 h.p., B runs at 75 rev. per min., whereas A runs at only 72 rev. per min. As will be explained later, these are the "unit speeds" of these two wheels.

It should be clearly understood that this term, "high-speed," thus involves a consideration of power as well as speed. It is not related directly to the peripheral speed, although, as stated before, high peripheral speed is usually an attribute of a high-speed wheel. At the same time, it may also be an attribute of a low-speed wheel, and therefore it is not a proper criterion of speed in the sense in which it is here used. Wheel No. 1800 illustrates this point very well. At the normal speed of this wheel, the coefficient of peripheral speed, $\phi = 0.81$. This value is as high as for any of the other wheels, and yet this wheel is lower in speed than all the others, except No. 1795.

Unit Speed.—The "unit speed," N_u, used throughout this paper, is in all cases the speed, under 1 ft. head, of a wheel of the same design as the one under consideration, and of such size as to develop 1 h.p. at full gate. Other units might be used and serve the same purpose of comparison, but in America these are the most convenient. In Europe, under the metric system, the practice is to reduce to one metric horsepower, under 1 m. head. The result is termed the "specific speed," and is usually designated by the symbol, N_s.

The general equation for N_u is developed as follows:

N = speed of any runner,
P = power of same,
D = diameter of same,
h = head on same,
h_u = unit of head,
P_u = unit of power,
N_u = unit of speed,
N' = speed of above runner under unit head,
P' = power of above runner under unit head,
D_u = diameter of homologous runner developing unit power under unit head.

Then

$$N' : N = \sqrt{h_u} : \sqrt{h} \qquad N' = N\sqrt{\frac{h_u}{h}} \quad \ldots\ldots (a)$$

$$P' : P = h_u^{\frac{3}{2}} : h^{\frac{3}{2}} \qquad P' = P\left(\frac{h_u}{h}\right)^{\frac{3}{2}} \quad \ldots\ldots (b)$$

$$P' : P_u = D^2 : D_u^2 \qquad D_u = D\sqrt{\frac{P_u}{P'}} \quad \ldots\ldots (c)$$

$$\frac{N'}{N_u} = \frac{D_u}{D} \quad \ldots\ldots (d)$$

Substituting Equations a, b, and c in Equation d.

$$\frac{N\sqrt{\frac{h_u}{h}}}{N_u} = \frac{D\sqrt{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}}{D} = \sqrt{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}$$

Simplifying,

$$N_u = \frac{N \times \sqrt{P} \times h_u^{\frac{5}{4}}}{P_u \times h^{\frac{5}{4}}} \quad \ldots\ldots (1)$$

With $P_u = 1$ h.p. and $h_u = 1$ ft. this becomes

$$N_u = \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \quad \ldots\ldots (2)$$

Given the power and speed of a wheel under any head, the unit speed may be determined by substituting in Equation 2. It is not

necessary to go through the process of double reduction as was done in comparing wheels A and B. For example, take wheel A, just considered.

Here

$$h = 100 \qquad P = 2\,080 \qquad N = 500$$

Substituting in Equation 2,

$$N_u = \frac{500 \times \sqrt{2\,080}}{100^{\frac{5}{4}}} = 72$$

which checks the former computation. Under the metric system, $P_u = 1.01$ h.p. and $h_u = 3.28$ ft. Substituting in Equation 1,

$$N_s = \frac{N \times \sqrt{P} \times 3.28^{\frac{5}{4}}}{1.01 \times h^{\frac{5}{4}}} = 4.46 \times \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \qquad (3)$$

or

$$N_s = 4.46\, N_u.$$

Hence, to transform a unit speed from the English to the metric system, multiply N_u by the coefficient, 4.46.

This method of classifying wheels according to their unit speeds is one of great convenience. Suppose, for example, that it is desired to equip a plant with units using runners having a maximum capacity of 1 000 h.p., each at 200 rev. per min., under 40 ft. head. From Equation 2, $N_u = 63.3$, and therefore only wheels approximating this unit speed need be considered.

Again, suppose that the sizes of the units and the speed for a prospective plant under 100 ft. head have not been determined. It is desired to know what results could be obtained with a certain runner, the characteristics of which are well suited to the proposed service. The unit speed of this runner is 70. Substituting in Equation 2,

$$70 = \frac{N \times \sqrt{P}}{100^{\frac{5}{4}}} \qquad N \times \sqrt{P} = 22\,120.$$

This type of wheel will give any combination of power and speed, whatsoever, that will satisfy this equation. With the inverted scale of a duplex slide-rule, innumerable combinations of power and speed may be read off as rapidly as one can move the runner. This is a great convenience in preliminary calculations. In the above case, for

instance, any of the following combinations of power and speed can be produced, depending on the size of the runner used.

1 000	h.p.	at 700	rev. per min.
1 460	"	at 600	"
1 960	"	at 500	"
3 060	"	at 400	"
5 450	"	at 300	"
12 200	"	at 200	"
49 000	"	at 100	"

Classification According to Unit Speeds.—Some method of classifying turbine runners on the basis of the duty which they are able to perform is essential to an intelligent study of turbine characteristics. It is obviously futile to compare the relative merits of two wheels which cannot accomplish the same work. They are in different classes, and, as such, might naturally be expected to possess different characteristics.

Since the unit speed is a measure of complete performance, both as to speed and power, it supplies an admirable basis for the desired classification. In this it should be noted that such a classification is independent of the details of design. The angles of the vanes, general proportions, peripheral speed, etc., may all be different, and yet the wheels are classified as of the same type if they have the same unit speed, because they, and they alone, are capable of doing the same work.

In Europe it is the practice to classify runners according to their coefficients of peripheral speed. A "low-speed" runner has a value of ϕ equal to about 0.6, a "normal-speed" runner, about 0.7 and a "high-speed" runner, about 0.8. These distinctions, however, embody the element of capacity as well as speed, although in an indirect and rather indefinite way. This is because a value of $\phi = 0.6$ is not ordinarily used, except with wheels of low capacity, 0.7 with those of normal capacity, and 0.8 with those of high capacity. The distinguishing element of capacity is really there, although its influence is not definitely measured. Such a method, of course, cannot be as exact as that based on unit speeds. In the latter case, both capacity and speed are definite mathematical factors.

Influence of ϕ Upon the Characteristic Curves.—The extent to which changes in the value of ϕ at normal speed affect the efficiency curves of turbine runners is difficult to determine, because, with wheels of rational design, the value of ϕ, as just stated, keeps step with the variation in the capacity of the wheel. It is, of course, a principle of economical design that a runner shall be made no larger in diameter than necessary. For a given speed, the lower the value of ϕ the smaller the wheel, and hence a low value of ϕ is suitable to a low-capacity wheel, and a high value to a wheel of large capacity. It may be generally stated as a principle of rational design that ϕ should always be made as low as possible, and that it should be forced up to high values only in order to get additional capacity.

Wheel No. 1800 was an experimental runner designed contrary to this principle for purposes of investigation. It is of irrational design in that it is a wheel of comparatively low capacity with a high value of ϕ. The value of ϕ for a wheel of this capacity need not be over 0.7 at normal speed, whereas it was 0.81, according to test.

The results obtained with this wheel appear to indicate that, with wheels of similar capacity, the value of ϕ at normal speed has no very important effect on the characteristic curves, although a high value of ϕ does show some tendency to pull down the efficiency. Fig. 19 shows that the three wheels of medium and low capacity gave similar results at normal speed, although the values of ϕ were 0.70, 0.76, and 0.81, respectively. The three wheels, however, which had the same value of ϕ, namely, 0.81, but differed considerably in capacity, ranging from medium power to the extreme limit of high power, gave widely different curves. All of which seems to indicate that forced capacity is the chief factor which causes the variation.

A comparison of Figs. 14, 17, and 18 shows a further similarity between the three wheels having different values of ϕ. The variation in efficiency at part gates, due to a variation of 0.1 in ϕ both above and below its normal value, is quite similar in all three cases. In fact, what difference there is between the three wheels is in favor of the high value $\phi = 0.81$. This wheel shows less drop in efficiency at part gates, due to an increase in speed, than either of the others.

In view of this evidence, and for lack of any experimental proof to the contrary, the writer believes it safe to say that the efficiency is not a function of ϕ, to any considerable extent.

Principal Difference Between Low-Speed and High-Speed Wheels.—Before proceeding further, it may be well to state that the deductions made from these tests should not be given too strict an application. Results necessarily vary with changes in design, and, while the results obtained with these wheels are of a nature broadly representative of other wheels of similar capacity, nevertheless, considerable variation in individual cases must be expected.

The most significant fact to be observed, in connection with the efficiency-speed curves of these wheels at various gates (Figs. 4, 6, 8, 10, and 12) is that all of them peak at different speeds. The most efficient speed of the wheel varies with the gate opening, increasing as the latter increases. This is true of all reaction turbines, whether of low or high speed, but not in the same degree. In the case of high-speed wheels, the difference between the best low-gate speed and the best full-gate speed is much greater than in the case of medium-speed and low-speed wheels. Table 6 shows the comparison between 0.4 gate and full gate for all the wheels under 1 ft. head, listed in the order of their capacity.

TABLE 6.

Wheel No.	1795	1800	1799	1797	1796
Most efficient full-gate speed	40	52	48	58	64
Most efficient 0.4-gate speed	31	41	38	35	35
Percentage of drop in speed	22.5	21.1	20.8	39.6	45.3
N_u	35.2	56.7	59.0	72.3	78.5

It may be observed from Table 6 that the drop for low-speed wheel No. 1795 and for the two medium-speed wheels, Nos. 1799 and 1800, is about the same, whereas it is nearly twice as great for the high-speed wheel No. 1797, and even greater for the extremely high-speed wheel No. 1796. This drop does not seem to vary much with the capacity of the wheel for values of N_u below 60, approximately. Above $N_u = 60$, or thereabouts, the drop increases rapidly as N_u increases.

When operating at constant speed, the inevitable result of this variation in the most efficient speed of the wheel is a sacrifice of efficiency at all gates except one. For example, if a speed is selected which will give the highest efficiency which it is possible to obtain at 0.75 gate, that speed will be too high to give the best obtainable efficiency at the lower gates and too low to give the best results at the higher gates. The best speed at which to run a wheel, therefore,

depends on which gate opening is the most important as regards efficiency. The best speed is the one which strikes the peak of the efficiency curve for that gate.

Naturally, the less variation there is in the best speed from low gate to full gate the less sacrifice of efficiency there will be when the wheel is run at constant speed. It has just been shown that this variation is much less with wheels of low and moderate unit speeds than with those of high unit speed, and therein lies the most important difference between these several types.

As an illustration of this point, compare low-speed wheel No. 1795 with high-speed wheel No. 1796 (Figs. 4 and 6). The maximum efficiency of the former at 0.444 gate is 82%, occurring at a speed of 34.5 rev. per min. The efficiency at the normal speed of 40 rev. per min., however, is 80%, and hence 2% has been sacrificed at this gate to the requirement of constant speed.

In the case of No. 1796, the maximum efficiency at 0.462 gate is 74.8%, occurring at 37.8 rev. per min. At normal speed of 53 rev. per min. the efficiency is only 62.5% and about 12% has been sacrificed to the requirement of constant speed. In addition to this, the peak of the curve of No. 1795 is about 7% higher than that of No. 1796, and hence the total difference in favor of the low-speed wheel at this gate is about 17 per cent. Fig. 19 shows graphically what a wide difference exists between the efficiency curves of medium-speed and high-speed wheels. The relatively poor showing of the two high-speed wheels, Nos. 1796 and 1797, is due to the two characteristic deficiencies just alluded to, namely, rapid drop of the efficiency-speed curves at part gate, and relatively lower maximum efficiencies at all gates. The latter condition is theoretically inevitable. It is impossible to discharge a large quantity of water through a wheel of given size as efficiently as a moderate quantity.

Another peculiarity which is distinctly characteristic of high-speed wheels is that the power at the high gates increases as the speed increases beyond normal. By over-gating the wheel it is possible to hold up a substantial increase of power with good efficiency at extremely high values of ϕ. For example, wheel No. 1796 (Fig. 6), at 1.077 gate, increases constantly in power and efficiency until ϕ has reached a value of 1.00. No. 1797 (Fig. 8), at 1.00 gate, increases until $\phi = 0.97$.

This feature of high-speed wheels is a valuable one, because it fits them especially for the only work for which they are at all adapted, namely, low heads. Low-head plants are almost invariably subject to relatively large reductions of head, due to back-water in the tail-race. If the head drops very much, the value of ϕ necessary to maintain constant speed will run up very high. A wheel which gives more power at a high value of ϕ than it does at the normal value, is a great advantage in such a case, because it increases the capacity of the turbine by just that much at a time when the power is perilously deficient on account of the reduced head. At such a time, it is necessary to open the turbine gates farther than would be advisable under the normal head. Under the latter, such over-gating would be wasteful, because the increase in power is very small and not at all porportionate to the increase in discharge. With some wheels, the power is even less than it was at the previous gate, on account of excessive drop in efficiency. Under the reduced head, however, with high values of ϕ, the increase in power is qute considerable. With No. 1796 (Fig. 6), for instance, the increase in power from 1.00 gate to 1.077 gate, at normal speed, is practically nothing, due to a 4% drop in efficiency. At $\phi = 1.00$, however, the increase in power is about 20%, and the efficiency is about 6% better. The over-gate, therefore, should be used only under a reduced head.

Reference to Figs. 4, 10, and 12 will show that the wheels of lower unit speed do not possess this characteristic. In every case, the high-gate curves fall off at the high values of ϕ.

Considerations Affecting the Choice of Wheels for Low Heads.— It is futile, of course, to attempt to define within fixed limits the heads which should be included in this classification. Some plants under moderately high heads operate under conditions very similar to those which are distinctly characteristic of the typical low-head plant. Others under low heads possess some of the characteristics common to high heads. If any useful deductions are to be drawn from such a study, they must be based on a consideration of typical conditions, with a liberal allowance for the inevitable exceptions which require special treatment.

A typical low-head plant may be illustrated by an installation where the head is created by building a dam across a large river flowing through a comparatively flat country. In such cases, it is im-

practicable to build a high dam, because the water cannot be confined behind it.

Having, probably, little storage area, other than the bed of the stream, such a power cannot usually be developed much beyond the minimum flow of the stream. If it were possible to impound the water which is not used during light loads for use during peak loads, then the part-gate efficiency of the turbines for such a plant would be of importance; but, usually, the water which is not used during light loads goes over the dam, and therefore the efficiency of the turbines at such times is relatively unimportant. Under such conditions, the poor part-gate efficiency of high-speed wheels is of no importance.

The important considerations are high efficiency at normal gate, as affecting the normal quantity of power which can be developed from the available flow; high speed, as affecting the cost of the electrical machinery and, to a limited extent, the cost of the turbines; high power at constant speed under reduced heads due to back-water, which is the inevitable coincident of flood conditions.

In respect to the first consideration, the high-speed wheel is somewhat deficient, depending on the extent to which the wheel is pushed for speed. Taking the efficiency of No. 1799, 90.43%, as the standard for a wheel of medium speed, the high-speed wheel No. 1797, yielding 87.39%, suffers about 3% by comparison. Wheel No. 1796, which is a more extreme type than No. 1797, gave 86.15%, or about 4¼% less.

Another point to be noted is that the maximum efficiency of high-speed wheels occurs at a higher gate opening than that of lower-speed wheels. With the latter it usually occurs at from 0.70 to 0.75 gate, whereas with the former it is usually from 0.80 to 0.85 gate. In order to provide for the customary overload of 25% with high-speed wheels, it is necessary that they shall carry the normal load at a lower gate than the most efficient one. This causes a further loss of efficiency.

In respect to the second and third considerations, the high-speed wheel has an overwhelming advantage, and is undoubtedly the best wheel for typical low-head service. Table 7 gives a comparison of the results which could be accomplished by the three types of runner, Nos. 1796, 1797, and 1799, respectively, if applied to a plant having a maximum capacity of 500 h.p. per runner under 20 ft. head. Low head is assumed to be 13 ft.

TABLE 7.

Wheel. No.	20 Feet.			13 Feet.			N_u
	Horse-power.	Revolutions per minute.	Efficiency.	Horse-power.	Revolutions per minute.	Efficiency.	
1796	500	149	82.0	279	149	80.0	78.5
1797	500	136	83.0	277	136	80.5	72.3
1799	500	112	82.7	246	112	79.5	59.0

Runners for Intermediate Heads.—It is as the head increases and back-water conditions become less important, and as the speed of the ordinary runner, increasing with the head, more nearly approaches an economical value, that the use of the high-speed runner becomes more questionable. As these conditions change, the considerations which favor high-speed wheels gradually disappear, and others of more importance become dominant.

As the head increases the storage capacity usually increases, and, with the capacity to impound water for peak loads, the part-load economy of the turbines becomes highly important, and the medium-speed wheel with its high efficiency is clearly indicated. The higher the head the more valuable a cubic foot of water becomes, and the more important it is to save it. High speed becomes a subordinate consideration. If the additional power which can be developed by the lower-speed wheels with their high efficiency is worth more than the interest on the saving in electrical apparatus which would result from the use of high-speed wheels, then the former are certainly the more economical proposition. This principle, of course, is fundamental and commonly recognized, but is not always intelligently applied for lack of specific knowledge regarding the disadvantages incidental to high speed. If purchasers were as well informed as they ought to be, as to the deficiencies of high-speed runners, there would be fewer of them in use at the present time. It is hoped that this paper may supply some welcome information.

Table 8 shows comparative efficiencies at various gates for medium-speed wheel No. 1799 and high-speed wheel No. 1796. In view of these figures, a very weighty reason would be required to justify the use of No. 1796 instead of No. 1799, if efficiency is of any importance.

High-Head Wheels.—Runners for high heads pass through all the intermediate stages of design between the mixed-flow wheel and the

purely inward-flow or so-called Francis type, according to the head available and the power developed. As the head increases the unit speed tends to decrease, and the design approaches that of the low-capacity Francis type (No. 1795). Although the designs of these wheels vary considerably, the characteristic curves vary but slightly from the best results of medium-speed wheels.

TABLE 8.

Wheel No.	Gate Opening.									
	0.30	0.40	0.50	0.60	0.70	0.75	0.80	0.85	0.90	1.00
1799	68.5	76.5	82.0	86.0	88.0	90.4	90.0	89.2	88.0	83.0
1796	41.0	55.5	66.0	73.5	79.5	82.0	84.5	86.2	85.0	82.0

The efficiency curves of wheel No. 1795 (Fig. 5) at normal speed, with $N_u = 35.2$, and wheel No. 1799 (Fig. 11), with $N_u = 59$, are very similar. On account of the discrepancy between the peaks of these two curves, the part-gate efficiencies are about the same (Fig. 19). Otherwise, the latter would be a trifle higher for the low-speed wheel. The general tendency is for the efficiency, particularly at low gates, to improve as N_u decreases.

The reason wheel No. 1795 did not show as high maximum efficiency as No. 1799 is that, being a low-speed wheel, it developed much less power under test, and hence the loss, due to mechanical friction, was a much greater percentage of the total power developed than was the case with No. 1799. Wheel No. 1795, operating under a head of several hundred feet, particularly in large diameters, would undoubtedly develop 90% efficiency or better. The effect of mechanical friction, which has been discussed at length, is especially detrimental to the efficiency of a low-power wheel such as this, when tested at Holyoke. No high-head wheel can by any possibility show as high efficiency in the Holyoke flume as it would in regular operation, assuming, of course, that in the latter case it is properly installed.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.

Discussion.*

By Messrs. Clinton S. Bissell, F. Lavis, E. W. Lewis, and George L. Dillman.

Mr. Bissell. Clinton S. Bissell, M. Am. Soc. C. E. (by letter).—The author states specifically that the paper is not a theoretical discussion of railroad location, and later he cites as the "desired results":

1.—Easiest grades possible, against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line, etc.

He also mentions ruling grades of 0.3% on the Little Kanawha Division, and 0.5% on the Burnsville and Eastern Division. The momentum grade at Mile 20 on the latter division, is introduced "to avoid a long détour"; and, in fact, the reader is impressed throughout the paper by the evident great advantage of the "shortest line."

To the writer, a railroad is simply an industrial plant, instituted to furnish transportation, and the "desired results" to be realized, are embodied in the one problem, viz., to obtain the line of greatest efficiency. The author has mentioned no measure of efficiency, nor has he indicated any limitations as to the duty required of the line. Without these, the writer is at a loss to understand the basis upon which the location was made, unless it was merely to get a tolerably direct line, with gradients as low as good direction would allow. The author has either disregarded or withheld one element which Wellington

* This discussion (of the paper by W. S. McFetridge, M. Am. Soc. C. E., printed in *Proceedings* for April, 1909, and presented at the meeting of May 19th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

rightly cites* as of "overwhelming importance," namely, the probable amount of traffic. The second important element—that of easy gradients—the author has observed without correlation to the probable traffic. The other two elements—curvature and distance—are, within reasonable limits, minor details of location, and yet apparently the author has given them great attention, to the negligence of one of the two more important elements. Mr. Bissell.

Since nothing can well be further removed from the theoretical side of the problem of location than the actual dollars and cents which the road can either earn or save, let us consider such part of the profile as the author has shown in Fig. 3, between Burnsville and Buckhannon.

From Buckhannon westward it is noticeable that the two stretches of ruling 0.5% grade amount to about 3 miles in 33, or only 10% of the distance, neglecting the momentum grade which, according to the author, can be removed.

Assuming at random that the line will carry annually 3 000 000 tons of freight westward, with heavy locomotives and modern high-efficiency cars, the 0.5% ruling grade permits a train load of 1 700 tons of lading; and the same train, returning with about one-third as much lading, can surmount the 1% grades going east. Taking the train-mile cost at $1, for convenience, the annual cost of operation can be roughly estimated.

It is rather surprising that the two ruling 0.5% grades were not made 0.3% to accord with those on the Little Kanawha Division. For the sake of example, it will be assumed that this could have been done at the expense of 4 miles more of distance at a gross cost of $50 000 per mile. For the 0.3% grade the train load going west would be 2 420 tons of lading; and the same train would return east with about one-fifth as much lading, against the 1% grades. Hence, the following figures may be deduced:

3 000 000 ÷ 1 700 × 33 × $1	west-bound	$58 240
Same trains returning	east-bound	58 240
Annual operating expenses on 0.5% grades		$116 480
3 000 000 ÷ 2 420 × 37 × $1	west-bound	$45 870
Same trains returning	east-bound	45 870
Annual operating expenses on 0.3% grades		$91 740
Difference		$24 740
Less interest at 4% on cost of 4 miles of line		8 000
Annual saving		$16 740

* "The Economic Theory of Railway Location," by the late A. M. Wellington, M. Am. Soc. C. E.

Mr. Bissell.

At 4%, this saving justifies a present expenditure of $418 500 to reduce the ruling grades from 0.5% to 0.3%; this reduction increases the efficiency of the line by a saving of 14% in expenses; and probably there would be enough capital left to take out the momentum grade at Mile 20.

Unless the matter of location be viewed from some such standpoint as this, the writer does not understand how the "shortest line" practicable on the "easiest grades possible" and on the "lighest curvature" can be decided. The intimate correlation of these three elements does not permit of such a decision, except upon some basis other than simply the contour of the country.

A few years ago the writer made several surveys and locations along the south shore of Nova Scotia between Cape Sable and Shelburne. The drainage, flowing down to the sea, crossed the direction of survey about at right angles. The route along the seashore was too irregular to justify a low line for any considerable distance, and a high line several miles inland touched but few towns, because these, for the most part, were situated along the shore. The final finished location was a compromise between these two lines. The whole route was wooded, especially on the slopes of the ridges where the maximum grades occurred. The specifications of the Dominion and the Provincial Governments fixed the maximum grade at 1.5% and the maximum curvature at 6 degrees.

In conducting these surveys, the writer found it necessary to take contours only on the ruling grades and a few other points on the line. The grades were first cut through, and the profile taken, after the manner mentioned in this paper, from the summit downward. To accomplish this object, the transit was frequently carried several miles ahead of the measured line and set in the summit or other governing point, and the profile run from a temporary bench-mark, the elevation of which was assumed or estimated with a barometer. The measured line was then brought forward, connected, and run again over the maximum grade. Particular attention was then given to the contours and to the topography of the stream crossing which was usually found at the foot of the grade. The great advantage in this method is the fixing of the proper point of connection at the foot of the grade with the assurance of passing through the governing point ahead. The writer believes that notwithstanding the necessity of running over the grade twice, time is saved by avoiding much backing up and subsequent alteration of the line back of the foot of the grade. It gives a continuous chainage, saves much contouring, and produces a preliminary map which lies close to the location at all the critical points.

The author's figures on costs are very interesting and valuable; but

the maximum rate of progress mentioned, 7 miles of preliminary line and 4½ miles of location per day, has never been equaled by the writer. He considers it excellent work for one party to accomplish even half of these distances in one day. Mr. Bissell.

F. Lavis, M. Am. Soc. C. E.—This paper is a valuable addition to the somewhat scanty literature relating to the subject, and, undoubtedly, will be of considerable interest to those connected in any way with the conduct of surveys for the location of railroads. It is especially so to the speaker, as the methods used agree very closely with those developed by him between 1898 and 1902 on the Choctaw, Oklahoma and Gulf Railroad.* Those surveys resulted in the construction of more than 800 miles of railroad, and, probably, at least as many more miles of location were actually staked out on lines which were not built. The speaker has no exact figures at hand, but it is probable that more than 5 000 miles of preliminary were run during that time. Mr. Lavis.

During the last ten or fifteen years these general methods, with slight minor variations to suit local conditions and individual ideas, have become fairly well recognized as standard on most of the larger railroad systems of the country; at any rate, those engaged to any extent in building new lines and extensions, recognize the absolute necessity of such methods, in order to determine without doubt that the line finally adopted is such "that no other line can be built through the same country, with the same or better ruling grades, with less expenditure, at the same unit prices."†

The data relating to costs show close agreement with the costs presented by the speaker in the paper referred to, and, although, in both instances, they are somewhat higher than a great many locating engineers think their surveys have cost, it will generally be found that, when these low costs are given, the fact has been lost sight of, that on any survey for the location of a railroad, the costs, both of the field parties, and a proper proportion of the expenses of the main and division offices, covering all work from the reconnaissance to the completion of the final maps and profiles, ready for the contractors to bid on the construction, and the real estate agents to purchase right of way, should be included.

The surveys described by Mr. McFetridge were made in 1902 and later; the costs previously given by the speaker were for surveys made during the latter half of 1902, so that the time agrees fairly closely. During 1903, of course, it was easier to get men; and salaries, as a rule, were comparatively low, so that the costs, during the latter part of the time when these surveys on the Little Kanawha were made,

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 104 (1905).

† "Railroad Location Surveys and Estimates," p. 8.

Mr. Lavis. would tend to be low rather than otherwise; this is shown also by the salary list.

Comparing the costs with those presented by the speaker,* it may be noted that the costs of the field work of the preliminary and location lines agree very closely. The average cost of the lines, as given in Table 2, shows the preliminary to have been about $28, and the location $88 per mile, whereas the costs given by the speaker were $26 and $71, respectively. The costs of the completed location, as described in the paper, vary from $127 to $188 per mile, averaging $151, on the basis of the mileage of located line staked out; or from $171 to $251, averaging $202, if the mileage of adopted location be taken, which latter may be compared with the cost of $192 per mile of adopted location as given by the speaker. The costs of field parties per day are given by Mr. McFetridge as between $35 and $40, whereas those described by the speaker cost from $42 to $65, and averaged $46. The somewhat lower cost per day of the field parties on the Little Kanawha, although the cost per mile of line was higher, is probably due to the difference in organization. On the Choctaw, although all lines were subject to revision, of course, by the Chief Assistant Engineer and the Chief Engineer, the Locating Engineers, whose whole expense, in addition to that of an Assistant Locator, was charged to the field party, were expected to be able to decide questions which, on the Little Kanawha Railroad, were apparently not left to be settled by the Assistant Engineer in charge in the field.

Judging from the description of the country and the number and length of preliminary lines run, it would appear that the two surveys are fairly comparable. The country described by Mr. McFetridge is in many cases rougher in detail than that through which the surveys described by the speaker were made, though, in the latter case, the character of the country required the detailed examination of a much wider range in order to determine the best location. This is shown by a comparison of the mileage of preliminary with that of location; on the Little Kanawha lines there were 2.37 miles of preliminary to 1 mile of location, whereas on the lines described by the speaker, the average was 3.14, a little greater than 30% more. The effect of the rugged detail is shown in the considerably increased cost of staking out the located lines on the Little Kanawha.

The generally prevailing idea as to the cost of railroad surveys is, that in the United States it should be about $100 per mile; and in a review† of the speaker's "Railroad Location Surveys and Estimates," a well-known railroad engineer made the statement that "much excellent location has been made in America in not easy country for less than $50 per mile of located line, and this cost includes the needed pre-

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 133.

† *Technical Literature*, April, 1907, p. 175.

liminary for that located line." He adds, referring to the supposed high costs of the Choctaw surveys, "We must conclude * * * that the parties were not made up of veterans." Veterans or not, the speaker does not believe that any line, outside of a tangent across a prairie, and sometimes not even that, can be located and proper maps completed, for any such sum as he mentions, and that for any ordinary country a rough estimate of $200 per mile would be much nearer correct for properly conducted surveys and the preparation of a proper set of plans ready for construction, these latter, of course, not including details of structures. The trouble with most of the published costs of location surveys is that they do not include anything but the cost of staking out a location on the ground; and supervision, office expenses, equipment, costs of preparation of proper maps, profiles, etc., are omitted. Mr. Lavis.

There is considerable difference, also, in the amount and kind of information developed by surveys. As an instance of a not uncommon form presented to a contractor as the location profile, Fig. 12 shows a part of a profile of a line in the Southwest, which was actually sent to a New York contractor, and on which he was asked to make his bid, this bid to be a lump sum for the whole line. Fig. 13 shows a section of a typical profile of located lines such as is usually required by most of the railroads in the West. The material in the cuts is explored by making test pits and soundings, as are also the foundations for structures, when necessary, and the information obtained is shown on the profile. The quantities in cuts and fills are given, with the approximate length and location of the haul; the quantities in structures are estimated and shown. In short, the profile contains all the information necessary to prepare a close estimate of cost, and gives the contractor all the information, as to the nature of the work, which he requires in order to make an intelligent bid.

The profile shown by Mr. McFetridge in Fig. 9, as the standard form for a location profile, seems to the speaker to be defective, inasmuch as it shows no quantities, gives little indication of the material to be excavated, or the amount or direction of haul, and no indication of the form or location of bridges, culverts, or other structures, or the quantities of material required to build them. The information required for the proper design of the larger bridges is shown in good shape on the situation plan, Fig. 10, but it is very desirable to show on the profile the type of structure and the space it occupies.

The speaker believes it to be of the greatest importance, not only to show the distribution of quantities on the location profile, but also to insist on their being worked out on the profiles of all projected locations. There is nothing which gives a man as completely the intimate knowledge of the line which it is really necessary for him to have as to work out the distribution, and get a clear idea of what he is to do

Mr. Lavis.

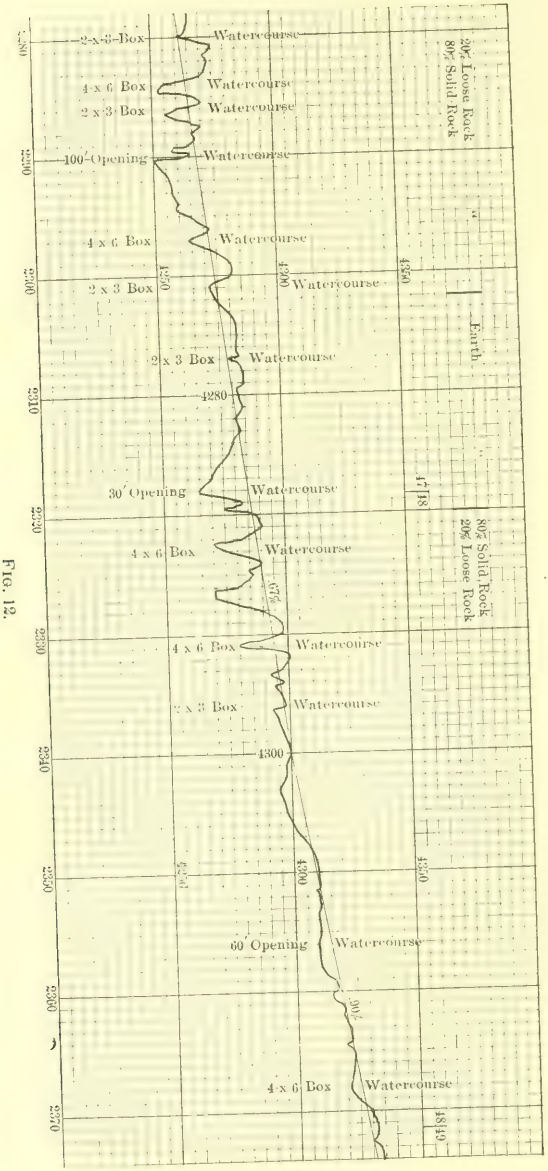

FIG. 12.

Mr. Lavis.

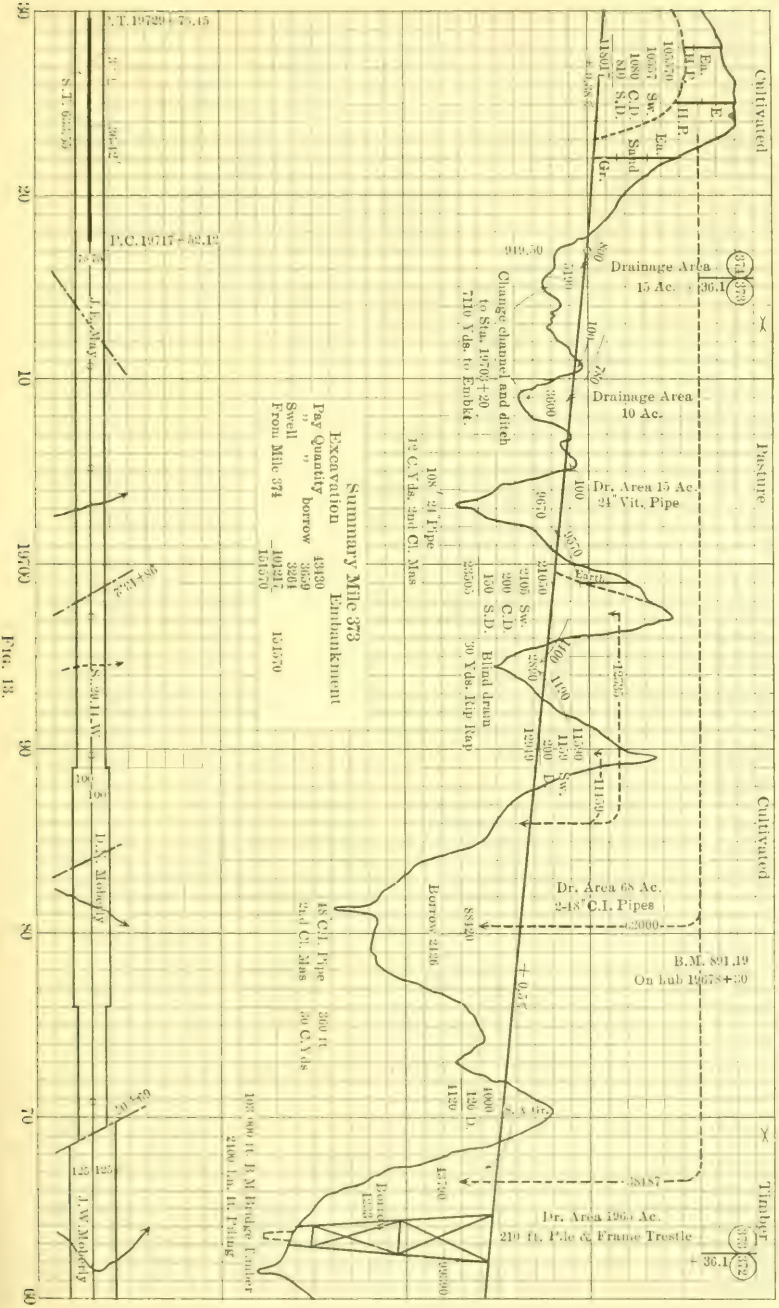

Fig. 13.

Mr. Lavis. with the excavation, and where he is to get material for the embankments.

One very important rule in regard to the final location, the speaker believes, should be made inflexible: After the profile of the located line has been platted, a temporary grade line fixed, tentative determinations of bridges, culverts, etc., made, and preliminary quantities calculated and distributed, this latter being most important, the Locating Engineer should take the profile into the field, and, walking over the line, should study carefully the situation on the ground in the light of the information which he then has, which, by that time, should be almost complete. This final review, if conscientiously done by a competent man, will often save many thousands of dollars in the construction, and avoid costly mistakes.

Although, in general, the methods described by Mr. McFetridge correspond closely to those described in the speaker's paper, previously referred to, a few minor differences may be noted. In keeping level notes the speaker prefers to keep the rod readings on the turning points entirely separate from the rod readings for elevations. It seems hardly necessary to have columns for grade and cut-and-fill in the location level notes, these belong more properly in the level notebook of the Resident Engineer, in which he records the check levels run on assuming charge of his residency.

The titles of profiles are generally more convenient when placed on the outside at each end; even if put inside they should be at each end rather than in the upper left-hand corner. Profiles accumulate rapidly on location, and with the title inside and only at one end, the chances are that two-thirds of them will have to be entirely unrolled to find the one sought.

The question of the advantages of separate sheets as compared with rolled maps has been discussed too often for the speaker to hope to say anything new at this time. Mr. McFetridge, however, although he evidently prefers the separate-sheet method, recognizes the necessity of having a large rolled map of the whole line in order to get a comprehensive view of the whole situation. The speaker takes exception to his very positive statement that the separate-sheet method of obtaining topography is quicker and better than any other; to say the least, this is open to argument. The speaker, who has had some little experience, prefers the method described in his paper.

For small general maps of the country, on which—in addition to the principal roads, trails, streams, etc.—may be shown the various preliminary lines as well as the final location, the speaker prefers a uniform scale of 5 000 ft. to the inch. These maps are useful, not only to the field parties and in the office of the Chief Engineer during the location, but also in discussions and conferences with the officials of the road, and others who only require general information. They are also

of convenience to contractors in bidding on the work, in order to enable them to form a better idea of its accessibility, etc. They should always be prepared at the beginning of the survey, and information should be added as the survey progresses. Mr. Lavis.

The speaker hardly sees the necessity of staking out a location for the purpose of choosing between two apparently equal routes, or, as was done in this case, for comparing the lines on either side of the larger watercourses. If the preliminary lines have been run with any degree of skill, and the topography has been properly taken, there should be no difficulty in getting, from a projected location on the map, and an inspection of the ground where this projected line would lie, all the information necessary for the comparison of any line or lines, except under the most exceptional circumstances.

The salaries named seem to be somewhat lower than those usually paid. In the West it is not uncommon to have a locating engineer in charge, with an assistant who looks after the details of the field work, leaving the former free to explore the country thoroughly, and devote more attention to the office work and the projection of various locations than he otherwise could. With a conscientious locating engineer this is a very good arrangement, and allows a much more careful threshing out of the country than would otherwise be possible. The author states that, on the surveys described:

"The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence."

The man who is responsible for the work of the party in the field should be with it most of the time, and if this devolves on the man in charge, it keeps his nose too close to the grindstone to allow of a proper perspective from which to view the problem as a whole, to say nothing of taking up practically his whole time. A proper, careful, study of the many possible projections cannot be made at night by a man who has put in already 10 or 12 hours' hard work in the field, although it is really surprising how much work men on locating parties manage to get through and thrive on.

The salary of the topographer is given at $65 per month, and, without intending in any way to cast any reflections on the young men who filled these positions, this seems to be altogether too little for good men. Taking topography has always seemed to the speaker to be a very simple operation, and it really is, if one has any sort of instinct for it, but it is remarkable how few men seem to grasp the idea, even after considerable experience. The speaker believes the topographer should be paid only slightly less than the transitman.

Mr. Lavis. The fundamental idea of the proper principles governing the staking out of the located line is stated very concisely by the author in the third paragraph on page 320,* and this idea cannot be impressed too strongly on all connected with work of this class. "In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections." In other words, there is to be no slavish following of the projected location as a line on the map, but rather to reproduce, on the ground, the line which the studies of the map and the general situation have shown is there, and is the best.

As the speaker has already pointed out elsewhere,† it is just here that modern methods differ from the old. It is neither the old paper location, made without reference to conditions in the field, nor the work of the Genius, described by Mr. Whinery‡ in his discussion of the speaker's paper, who threw his curves at the hills, as the cowboy throws his rope at a steer, and sometimes hit them and sometimes missed. If he missed, so much the worse for the railroad. The modern method collects all the available data affecting the location, eliminates definitely the unavailable lines, projects on the map the line which best fulfills all the requirements of the situation, and then uses all the intelligence of the Genius in reproducing that line on the ground. No extraordinary mathematical calculations are required to lay out the located line from the projection, only plain common sense and locating skill.

No amount of accuracy in making the original surveys and platting the map can compensate for lack of intelligence in transferring the projection to the ground, and extreme refinement of accuracy is wasted, if proper intelligence is used.

The statement on page 314,* contains, in a very small compass, the essence of the requirements which should govern the conduct of all location surveys, a "most minute study of the land, a thorough knowledge of its character, the running of many lines, the comparison of many estimates," are absolutely necessary in every case. The Locating Engineer, and by that is meant the man really responsible for the location, should be thoroughly steeped in the details of the country, and the word steeped is used advisedly, in the sense that there should be time enough in all cases to allow the information acquired to be taken in, digested, and to permeate him through and through.

One other point brought out in the paper, which is most important, is that actual surveys of all possible routes were made; as the author points out, this does not mean that an indiscriminate topographical map of the whole country was made, or was necessary. In spite of all that has been written about eliminating certain routes by eye, and per-

* *Proceedings*, Am. Soc. C. E., for April, 1909.
† "Railroad Location Surveys and Estimates," p. 125.
‡ *Transactions*, Am. Soc. C. E., Vol. LIV, p. 143.

haps by a hand-level, there is no question that absolute proof, by actual survey, should be furnished to the people putting up the money, or, at any rate, to those responsible for its expenditure, that there is no doubt that the location adopted is the best possible under the given governing conditions. The ability and so-called "eye for country," ascribed to the older generation of locating engineers, which many of them undoubtedly possessed, are just as necessary to-day, but must be supplemented by scientific methods, adequate surveys, and much hard work; and this is necessary, not only in order to be able to furnish proof that the line adopted is the best, but generally in order to find this line. As stated also in the paragraph referred to, "On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river * * *; but * * * a thorough study * * * gave the question a different aspect." This experience has been duplicated many, many times, and in many cases the study was not made, and the railroad paid the bill later. Mr. Lavis.

Not many years ago it was not an uncommon thing to hear some of the Geniuses with "eye for country"—and not much of anything else —talk about the foolishness of trying to get a 0.5% line through a 1.0% country. The results obtained by these surveys are only another instance of the possibility and practicability of finding these lines, if one knows how to look for them. This description, of work actually accomplished, should be borne in mind by all who have to obtain money for such surveys, and, when necessary, should serve to stiffen their backbone to demand all that is necessary to carry out such surveys in an adequate manner.

The author is to be congratulated on the very clear and concise manner in which the information is presented, and his recognition and presentation of the many vital points which differentiate proper methods of making location surveys and getting results from the methods too often adopted, of simply stringing together a lot of tangents and curves over which a train can be run somehow or other, if the stockholders' money holds out long enough and no one else builds a line alongside over which trains can be operated at half the expense.

E. W. Lewis, M. Am. Soc. C. E.—This excellent paper is of much interest to engineers engaged in railroad location. The methods outlined by the author are very nearly the same as those used by the speaker in making many surveys in widely scattered sections of the country. The make-up of field parties is identical with that used by him on the Northern Pacific Railway, and quite similar to general practice. Mr. Lewis.

The method of plotting topography in the field, while advantageous in many respects, would be a difficult performance in a rainy country, such as the Pacific Northwest, where it rains almost every day for long periods during the winter season.

Mr. Lewis. The speaker prefers taking the notes in a topography field book. These books being ruled like cross-section paper, the contours can be plotted and the exact measured distance from the center line given for each 5- or 10-ft. contour, making it very easy to plot on a large map in the office. This plotting may be done in the evening, but, ordinarily, it is left until the next day. With a draftsman regularly employed to keep up the map, each day's field work may be mapped on the succeeding day.

The method of taking the contours, by measuring from the center line with a tape, and obtaining the difference of elevation with a hand-level, is in accord with the speaker's practice as being the best when an accurate contour map is desired. He has learned by sad experience that the saving of a little time in taking topography by pacing distances, etc., is often the cause of a much greater loss of time when the paper location is run out.

In regard to spiraling curves, if the line is likely to be a final location, particularly if the prospect is good for immediate construction, the speaker prefers to run in the spirals with the location, although, unquestionably, it saves considerable time not to do so.

The costs of surveys given in the paper, are interesting as showing the wide differences on various lines. The approximate costs of some surveys with which the speaker has been connected in various parts of the country may be of interest. On the north bank of the Columbia River, in Washington, over very rough, difficult ground, largely on steep slopes, with numerous short tunnels, where, in some instances, it was necessary to lower men down the cliffs with ropes in order to get the line staked, the cost for a stretch of 32 miles was about $325 per mile. In Massachusetts, through an average country, largely wooded, the cost of 20 miles of location was about $150 per mile. In North Dakota, 60 miles of location over a rolling prairie, and crossing one deep, broad valley, cost $110 per mile. In another section of the same State, over rolling prairie, the cost was $80 per mile. These figures are for the total cost of the final location, including all preliminary, trial, and abandoned lines.

Mr. Dillman. George L. Dillman, M. Am. Soc. C. E. (by letter).—The writer is impressed by the absolute lack of value of this paper to the Profession, the uselessness of the conclusions drawn, and the pernicious possibility of these conclusions being applied to some other case of railroad location. No two locations are alike, and hardly any two are comparable in time, cost, or value.

The writer was transitman on two locations which are worth noting as being extremes in his practice: The first was in Cottonwood Cañon, on the Grand River in Colorado, between Dotsero and Glenwood Springs. Afterward the Denver and Rio Grande built its line on the opposite side of the cañon. There was a great deal of clearing, and

the topography was such that direct measurement was often impossible, so that a great deal of triangulation for distance was necessary. This location cost more than $600 per mile. Mr. Dillman.

The second location was on the North Platte River, in Nebraska, extending more than 100 miles northwest from North Platte. The country was open, there was no clearing, the tangents averaged about 9 miles, and the curve angles between them were small. This location cost between $17 and $18 per mile.

These locations were each fully staked on the ground. The costs include maps, profiles, and computed estimates, as well as all preliminary work. The parties had about the same number of men and equipment, except that more axe-men were added to the Grand River party. Each party worked efficiently and sometimes for long hours. If there was any difference, the cheaper location was the more carefully made. Tracings of each map were filed with the Interior Department, accompanying applications for rights of way across Government land, and each location was ready for the construction crews.

These locations are probably not extremes; there may have been cheaper locations made; undoubtedly, there have been more expensive ones. What is desired is to bring out the fact that there is no possible connection between the costs of railroad locations; and that a full knowledge of the cost in one case cannot be applied to any other case.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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CAISSON DISEASE AND ITS PREVENTION.
Discussion.*

By Messrs. J. S. Haldane, Frederick L. Keays, Albert J. Loomis, William L. Saunders, Walton I. Aims, and Seward Erdman.

Dr. Haldane. Dr. J. S. Haldane, F. R. S. (by letter).—After very carefully reading Mr. Japp's paper, the writer considers it to be an extremely valuable contribution to a practical solution of the prevention of caisson disease. He clearly points out the difficulties involved in adopting without modification for all compressed-air work the plan of "stage decompression" worked out by Dr. Boycott, Lieutenant Damant, and the writer, which has been in successful use since 1907, for divers of the British Navy. A full account of the experimental evidence on which this plan is based will be found in the *Journal of Hygiene*, 1908, pp. 342-443. It will be seen from this paper that most of the experiments were made at very high pressures, such as 75 lb., and that they were carried out with a view to deep diving. It was for many reasons desirable to leave a good margin of safety; and, therefore, this margin was left, particularly as practical experience in deep diving was very limited, on account of other causes which the writer and his associates had just succeeded in removing.

Work in tunnels and caissons, however, is conducted at much lower pressures; practical experience is much more wide; and, thanks mainly to Mr. Moir, recompression is usually readily available. It becomes a question, therefore, whether the margin of safety which was allowed in decompression cannot be narrowed without appreciable danger. The new evidence which Mr. Japp has brought forward certainly seems to

*This discussion (of the paper by Henry Japp, M. Am. Soc. C. E., printed in *Proceedings* for April, 1909, and presented at the meeting of June 2d, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

indicate that this is the case. He records the fact that in the construction of the East River Tunnels 330 "seasoned" men were employed for 36 days in two 3-hour shifts at a pressure of 40 lb., and were decompressed by a modified form of stage-decompression in 48 min., without a single serious case of caisson disease occurring. Thus there must have been about 24 000 individual decompressions without any mishap. On the principles laid down in the writer's paper, Mr. Japp calculates that the maximum air-saturation left in any part of the bodies of these men at the end of decompression, corresponded to a pressure of 27 lb. The writer's calculations agree with this estimate. In view of these results the author proposes to shorten stage-decompression in such a way that in place of the 19 lb. of residual saturation which the writer has proposed, 25 lb. should be left in the case of men who are "seasoned" to the work. This would greatly shorten stage-decompression, and render it a much more easily practicable process, although the pains in joints, etc., which are the most easily produced and least serious symptoms of caisson disease, would not be prevented. Dr. Haldane.

The reason for the proposed 19 lb. is that occasional serious cases seem to occur at working pressures down to 20 lb., and that a fatal case is recorded at 23 lb. On the other hand, there can be little doubt that for the great majority of men a pressure of 25 lb. would not cause appreciable danger. Provided, therefore, that susceptible men can be excluded, Mr. Japp's proposal would seem to be consistent with reasonable safety.

Susceptible individuals can doubtless be excluded, to some extent, by a careful medical examination, with special regard to the state of the heart and blood vessels, and to any general debilitating cause, among which age must be included. Another very important factor is fatness. Dr. Boycott and Lieutenant Damant* have recently shown by experiments on animals that when the exposure to compressed air is prolonged, fatness increases the susceptibility to caisson disease. They decompressed 70 guinea pigs at a dangerous rate after exposure to a pressure of 100 lb. Exactly half of the animals died of caisson disease, and the other half were killed. The percentage of fat was then determined in the bodies of the two sets of animals, and was found to be much greater in those which had died of caisson disease, the average difference being in the proportion of 225 to 100. The caisson disease simply picked out the fatter animals. In consequence of these experiments, which confirmed in a most striking manner the conclusions which had been previously reached on theoretical grounds, all British naval divers are now disqualified at the medical examinations from deep diving if they are inclined to fatness.

While a medical examination will probably exclude most of the

* *Journal of Hygiene*, 1908, p. 445.

Dr. Haldane. more susceptible individuals, it still seems very doubtful whether all can be excluded in this way. With new men, therefore, it is desirable that preliminary trials should be made with short shifts, as was the practice in the East River Tunnel work. These men should be carefully watched after decompression, and promptly recompressed if they show any threatening symptom; and any man who has shown threatening symptoms should be rejected.

With these precautions, in addition to having a medical air-lock and keeping the men within reach of recompression for about an hour after they come out, the writer thinks that in all probability Mr. Japp's plan would prove successful. For casual visitors, the duration of stay could be limited so that on coming out the maximum saturation in their bodies should not exceed 19 lb.

The conditions vary so widely in different kinds of work in compressed air that it seems hardly desirable to lay down by legislation or otherwise any hard-and-fast general rules as to decompression. The writer, however, thinks that for any particular undertaking in compressed air, special rules, suitable to the particular circumstances, should be drawn up and strictly enforced. If these rules could also have some authoritative sanction, as, for instance, in the case of "Special Rules" under the Coal Mines Regulation Act in England, engineers and contractors would be relieved to a large extent of what at present must be a very uncomfortable responsibility.

Dr. Keays. Dr. Frederick L. Keays* (by letter).—The prevention of compressed-air illness is, of course, most important. It is likely that with the modern demands of engineering, even greater compressed-air undertakings will be projected in the future than in the past. Any means, therefore, which will add to the safety of the workmen, will not only benefit humanity at large, but will also permit of more extensive undertakings.

The three great factors in the causation of compressed-air illness are: The degree of pressure, the time under pressure, and the decompression period. The degree of pressure is, of course, the chief factor; and it may be stated that, other factors remaining the same, the higher the pressure, the greater will be the dangers of compressed-air illness. The degree of pressure which is to be used in any undertaking is not a matter of choice, but depends on the depth to which the work must be carried. The only means of preventing illness, then, which are left to the engineer are the regulations of shifts and of decompression periods. In the past it has been thought that the longer one stays under pressure, the greater will be the dangers of compressed-air illness. This is true, up to a certain point, namely, that at which there is complete saturation of the blood and body fluids with the gases of the

* Late Medical Director for S. Pearson and Son.

air. Dr. Haldane has made a valuable contribution to the subject of compressed-air illness by his work on the question of body saturation, and Mr. Japp has worked out Dr. Haldane's theory in a plain and satisfactory manner. Dr. Keays.

The body fluids are practically saturated at any given pressure in about three hours. This belief is based on the results of the work in the Pennsylvania East River Tunnels. From a study of the records, it was found that men working for periods of less than 3 hours were less likely to have "bends," and much less likely to have a serious illness than those who worked for 3 hours or longer. Furthermore, it was found that in comparing the results of 6-hour shifts—which, since they were divided into two 3-hour periods with an interval of 3 hours at normal pressure, really meant 3 hours' exposure—with the results of 8-hour shifts, at almost equal pressures and with corresponding rates of decompression, there were fully as many cases of illness with the former shifts as with the latter, and certainly more serious and fatal cases. A point which has not been sufficiently emphasized in the past is that the real danger in compressed-air work comes with or after decompression. For this reason the practice, which has been almost universally followed, of dividing the working shift into two periods, is a bad one, for it doubles the number of decompressions. The results would be better in tunnel and caisson work, if continuous shifts of 4, 6, or 8 hours were arranged to suit the pressures; say, 4 hours for pressures of from 42 to 50 lb. +, 6 hours for pressures of from 32 to 42 lb. +, and 8 hours for pressures below 32 lb. +, and if the decompression periods were increased to reasonably safe lengths of time.

The question of the decompression period is a difficult one to settle. In the first place it must be admitted that certain men have individual susceptibility to the effects of compressed air. Just what the reason for this susceptibility is has never been learned. The fact remains, however, that while one man may work even at a high pressure and decompress rapidly without any symptoms of illness, another, under even more favorable conditions of pressure, length of shift, and time of decompression, may succumb to sudden death from syncope and collapse, and the autopsy will show large quantities of free gas in the circulatory system. Among practical compressed-air workers there is a decided difference of opinion as to the necessary length of decompression periods, some even claiming that quick decompression is preferable. Nearly all men, however, who have dealt with the subject from the physiological and medical standpoint, have advocated fairly long times for decompression. The writer believes that lengthening the time of decompression is the most efficient means of reducing the number of cases of compressed-air illness, as well as the number of serious cases.

On account of the common occurrence of cases of delayed onset

Dr. Keays. of symptoms, it is not believed that lengthening the decompression to periods claimed by Bert, von Schrötter, Hill, and Haldane, to be safe, would prevent all cases of illness, or even serious and fatal cases. In the writer's experience a comparatively large number of men first begin to have symptoms of compressed-air illness several hours after decompression, in some cases after as many as 12 or 15 hours, and in a fairly large number, after 3 hours. In two fatal cases symptoms began to appear between 2 and 3 hours after decompression. Why these men should not have had symptoms, even if they had taken as long as 1 hour for decompression, the writer cannot see. He believes, however, that reasonably long decompression periods are efficient in reducing the number of cases of illness, disability, and death.

Just what are reasonably safe lengths for decompression periods have never been determined. In the work on the Pennsylvania East River Tunnels after April, 1906, a rate of decompression of at least 1 min. for each 2-lb. pressure was required, and quite generally followed. This seemed to cause less illness and a reduction in the number of serious cases, as compared with previous months when there were no rules as to the rate of decompression. Even this rate of decompression, which was longer than has been required in any compressed-air work in the United States, did not prevent serious and fatal cases. For this reason the writer thinks that even slower rates of decompression should be tried, for pressures above 28 lb. +, below which, this rate of 2 lb. per min. appears to be safe.

Mr. Japp has set forth Dr. Haldane's method of "stage" decompression, which is claimed by the latter to be more efficient in preventing illness than uniform rates of decompression of even longer times. The writer does not think that enough experimental work has been done to prove the efficiency of "stage" decompression; but its use in diving has proved very satisfactory, and experience with the extra bulkhead in tunnel work, which necessitates an interrupted decompression roughly corresponding to Dr. Haldane's method, would argue favorably for "stage" decompression.

If it were possible, in accordance with Dr. Haldane's suggestion, to arrange a large decompression chamber in which a pressure equal to one-half the actual tunnel pressure could be maintained, where the men could pass a long decompression period in comfort, and use the time for bathing and dressing and, perhaps, for eating, it would probably prove a valuable means of preventing illness, and, at the same time, permit of working with comparative safety under higher pressures than could now be attempted.

Medical supervision and examinations are important and are instrumental in preventing serious results from the effects of compressed air. Men with organic disease are certainly less able to withstand the effects of compressed air than normal individuals. New men are

certainly more susceptible to the effects of compressed air than old workmen. When high pressures are being used, a preliminary test of about 1½ hours under tunnel pressure for all new men, followed by a second medical examination, and short shifts of from 1½ to 2 hours for the first working period, are effectual means of guarding against serious results among new workmen. These means proved very valuable in the work on the Pennsylvania East River Tunnels. Dr. Keays.

Much remains to be learned in regard to the prevention of compressed-air illness. This knowledge can be gained only by experimental work with animals, supplemented by records of compressed-air undertakings. By adopting new methods which appear to be improvements upon the old, we may hope to arrive, finally, at a more definite knowledge of what is reasonably safe in the lengths of shifts and decompression periods.

Compressed-air work in tunnels and caissons, where pressures above 30 lb. + are used, will always remain a dangerous occupation in spite of all reasonable precautions, for one can hardly hope to prevent all cases of illness. Those cases due to small quantities of free gas are almost sure to happen, and a small proportion of them is likely to result in paralysis, which may cause permanent disability or death. Until the reason why certain men are susceptible to the effects of compressed air, can be learned, and such individuals can be recognized, cases of sudden death from large accumulations of free gas are almost certain to occur occasionally.

Dr. Albert J. Loomis.*—The author deserves great praise for the clear and lucid manner in which he has stated the ideas of those who have investigated and written on the subject of compressed-air diseases. Dr. Loomis.

But after a pretty wide experience, extending back over twenty years or more, with pressures ranging from 15 to 42 lb., gauge pressure, the speaker cannot agree with him, or his authorities, as to the cause of the disease under discussion. While admitting the "saturation theory" as laid down by Bert, von Schrötter, Haldane, and others, the speaker does not think that it is the cause of the so-called caisson disease, for the following reasons:

First.—As to the formation of bubbles of gas or air in the blood-vessels themselves: They have only been demonstrated in exceedingly high pressures—15 or 20 atmospheres—which, at the present time, are considered impracticable as working pressures. If these bubbles should occur in the blood-vessels of the men, working as they do and coming out as they do, there would never be any use for hospital locks, as the men would all die of suffocation before they could be gotten to them, owing to these minute bubbles becoming entrapped in the very small

* Chief Medical Officer, North River Division, Pennsylvania Tunnels and Hudson Companies.

Dr. Loomis. vessels or capillaries of the lungs, and thereby preventing the oxygenation of the blood, a condition which operating surgeons frequently meet, to their great regret.

Second.—If the symptoms were due to any change in the blood itself, there would be a general set attacking all parts of the body to a greater or lesser extent, which is the very thing that does not happen, but, on the contrary, they show themselves in a very few localities, and most of these localities are in the lower extremities, situations which any surgeon who is at all proficient in locating nervous lesions by their symptoms, would place without hesitation in the spinal cord, and in the lower portion of the cord.

On the other hand, there is practically but one set of symptoms, the so-called "staggers," which would lead physicians to look for a cause or lesion in the brain, a condition which is quite rare when compared with the great number of cases occurring in the lower extremities.

Should there be any change in the composition or condition of the blood, the brain would naturally be the first organ to show such change, as it is without doubt the most delicate organ of the whole anatomy.

In reference to the CO_2, the engineers of the North River Division of the Pennsylvania Tunnels have prepared a chart on which all the cases of compressed-air sickness have been plotted, and this chart shows that, on that work, the greatest number of cases occurred at a time when the CO_2 was only 1 part in 1 000, with a pressure of from 20 to 25 lb.; and, likewise, when the pressure was 28 lb. and the CO_2 was 2 parts in 1 000, there were no cases of caisson disease.

While the speaker is not prepared to place much stress on the CO_2, other than as an indication of the general ventilation, he does think that weather changes play a very great part in the causation of the disease, as do also the habits of living of those engaged in the work.

When it comes to the examination of men, the medical officer should fit himself for the work by a close study of the conditions under which the men are to work, and his power of rejection should be absolute. The selection of the men should be the duty of one man who should be responsible for their welfare while they are at work under pressure.

In substantiation of these opinions, the speaker desires to state, that in the work of the Hudson Companies, in the construction of the Hudson and Manhattan Tunnels, about 38 000 men have been examined, since 1902, for pressures of from 15 to 42 lb., with three deaths, and in the construction of the North River tubes of the Pennsylvania Tunnels about 7 000 men have been examined for pressures of 15 to 38 lb., with no deaths.

Mr. Saunders. WILLIAM L. SAUNDERS, M. AM. SOC. C. E.—This is a subject about which very little of practical value has been written, and the speaker is glad to note that Mr. Japp has gone into the matter in so much detail, both from theoretical and practical standpoints.

Caisson disease, or the "bends" as it is commonly called, does not differ from other diseases, in that doctors are very apt to disagree about it. Heretofore the subject has been treated in print mainly from a medical standpoint, various theories being advanced as to the cause of this disease, that is, whether it is due to mechanical or chemical action. The speaker remembers when the most plausible theory advanced for the cause of caisson disease was that the blood, which had been compressed and held in a state of tension, was suddenly released by decompression, creating a pain when coursing through the veins. This was illustrated by the statement that if a string is wrapped tightly around one finger and then suddenly unwound, the finger is temporarily in pain. The subject has been discussed so fully by Mr. Japp, however, and the theory of caisson disease has been so well established by the experiments to which he refers that it is now generally accepted that bubbles are the cause of the "bends." Mr. Saunders.

Compressed air is the speaker's hobby, and in his early experience in seeking information about it he has soared, like the eagle mentioned by Mr. Japp, to a height of 3½ miles, where he has felt the earmarks of the "bends," and has descended to great depths in mines and in a diving apparatus under the sea, where similar sensations have been felt.

Personal experience in a matter of this kind always fits one to judge better the causes as well as the effects, and it may add something to the interest in this matter to recite an experience which the speaker had some fourteen years ago in the old Hudson tunnel, now called the McAdoo tunnel. This tunnel was started by Mr. D. C. Haskin a little before, or about the time of, the Centennial Exposition in Philadelphia, in 1876. Mr. Haskin conceived the idea of driving a tunnel under the Hudson River by maintaining a chamber of compressed air to hold up the face. He used no shield, and as the material through which he worked was more or less unstable, he met with considerable difficulty and some lives were lost. He sank his entire fortune in the enterprise, the work being taken up later by English capitalists, who contracted with Messrs. Pearson and Son to finish the tunnel. At that time E. W. Moir, M. Am. Soc. C. E., was in charge of the work, and the experience which he gained there has no doubt helped him to design and carry through the recent tunnel construction under the East River, which Mr. Japp has so ably referred to and with which he has been connected in so important a position. During the administration of the Pearsons, the late R. P. Rothwell, M. Am. Soc. C. E., at that time editor of the *Engineering and Mining Journal*, and the speaker were invited to examine the tunnel for the purpose of making a report thereon. About an hour was spent under some 35 lb. absolute pressure. Coming out of the locks rather rapidly, on reaching the washroom, the speaker was taken with a severe attack

Mr. Saunders. of the "bends," the effect being like that of a paralysis of the lungs. A horizontal tank, resembling a common horizontal boiler, was at hand, and the patient was transferred to a bench of straw, the door was closed, compressed air was turned on, and the relief was instantaneous. A pressure valve, provided for the purpose of admitting and discharging pressure from the tank, was used to enable the prisoner to escape, but it was noticed that when the pressure fell only two or three points the "bends" returned, and it was only by the gradual expansion of the pressure, occupying about ½ hour, that the patient was able to open the door and walk out cured. The necessity for reducing the pressure gradually, as shown very graphically by the return of the "bends," indicates the value of the theory of gradual decompression. It also bears out the idea that time must be given to permit the escape of the little bubbles which accumulate in the body.

An interesting incident was noted in connection with this experience. When the pressure in the tank was lowered, through the discharge of air, a fog formed instantaneously, and as soon as air was let in and the pressure increased, this fog was as promptly dissipated. The speaker experimented with this for some time—manufacturing and annihilating fog. The lowering of pressure, of course, resulted in cooling the air, and, *vice versa,* increasing the pressure heated the air. As the air was cooled, fog was precipitated immediately when the temperature reached the dew point, the capacity of air for moisture depending entirely on volume and temperature, that is, a certain fixed volume of compressed air or free air can hold only a certain quantity of moisture at a certain temperature. If the temperature is raised, it will hold more water; if the temperature is lowered, the moisture must be precipitated first in fog, and later, in the form of rain.

Mr. Aims. WALTON I. AIMS, M. AM. SOC. C. E.—The literature on caisson disease is quite extensive. The causes of the disease now seem to be well determined, but difficulty is found in conforming practically with the medical rules and regulations for its prevention.

The age of the compressed-air worker has an important bearing on his ability to withstand the effects of too rapid decompression, but youth generally lacks experience, and, in difficult work, some compromise has to be made in drawing the age limit for entering the compression chamber. Dissipation acts strongly in undermining a man's resistance to the effects of compressed air, especially among workers of mature years, and even some temporary indisposition may occasion an attack in one who has previously been pronounced physically fit by a medical examiner.

In nearly all compressed-air work there is difficulty in compelling the workmen to take a reasonable time in the air-lock while decompressing. Believing that this is due largely to the extremely un-

comfortable conditions in the ordinary air-lock, a method for relieving the lock of the cold and fog when decompressing was used at the Battery-Brooklyn tunnel by introducing a constant flow of heated dry air while the lock was being exhausted. In addition to removing the cold and fog from the lock, this heated air also provided for ventilation while decompressing, a feature not found in the ordinary compressed-air lock. The comparative freedom from caisson disease at the Battery-Brooklyn tunnel, where practically no age limit was established, seemed to indicate the value of thus heating and ventilating the air-lock during the decompression period. Mr. Aims.

Dr. Seward Erdman.*—The speaker wishes to acknowledge his appreciation of Mr. Japp's instructive contribution to the literature of caisson disease, and feels that the general adoption of the suggested "Step Decompression Method" will aid much in robbing compressed-air work of its gravest dangers. Dr. Erdman.

The reduction, to terms and curves of mathematical precision, of the time and pressure elements, as shown on his charts, is most interesting; but it seems to the speaker that there will always be another factor to be reckoned with, one which cannot be reduced to mathematical terms, and yet which will continue to render uncertain the result of exposure to compressed air in each individual case.

This indeterminate factor may be called the "personal equation," and seems to be dependent on the functional efficiency of the circulatory and respiratory apparatus.

In the experience of all who have seen much of compressed-air work, cases are not infrequent where men, who are seasoned air workers, will work daily, for long periods of time, at fixed conditions of pressure, length of shift, and decompression time, without suffering any ill effects; but who come to work the next day, under exactly the same conditions, only to develop a severe or even fatal attack of "bends."

The decompression time, indicated on Mr. Japp's charts, can be controlled by valves from the air locks; but, for the individual, desaturation of the body tissues and the blood must take place through the lungs; therefore the lungs act as the "escape valve" for the individual.

In explanation of such accidents as are above cited, it is likely that this "escape valve" is temporarily clogged.

Desaturation of any given part of the body will be impeded by causes which slow the blood current in that area.

Thus passive congestion of the lungs, or the presence of mucus in the air spaces (as from a cold) may act as clogs to the "escape valve," thus increasing the time necessary for desaturation, and, under such conditions, the body does not rid itself of the gases in the average time, and an attack of "bends" results.

* Formerly a Medical Officer on the East River Tunnels Contract.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.

Discussion.*

By John C. Moses, M. Am. Soc. C. E.

Mr. Moses.

John C. Moses, M. Am. Soc. C. E. (by letter).—This investigation is to be welcomed as a forerunner of many more in which examinations of the actual conditions under working loads will be made. Tests to destruction, or to any point beyond the elastic limit, are of very doubtful value, and the practice of making them the basis for designing has been justified only by the supposed impracticability of making such tests as are described by the authors. Certainly nothing can be more illogical than to find that load, the first application of which will cause destruction, and then to assume that any arbitrary fraction of that load can be carried indefinitely with a proportional factor of safety. The absurdities of this method have become especially apparent when applied to such materials as reinforced concrete, but are none the less existent when applied to homogeneous materials like steel.

The difficulty of making the proper kind of tests does not alter the facts, and the proofs now presented that such tests are not impracticable are very encouraging. They will need to be many in number and to be made on structures when first erected and again on the same structures after they have been in use a while. They will need especially to be made on details of all kinds in order to eliminate the very large element of guesswork which now enters into this very important part of designing. Some of the authors' conclusions bear very significantly on this subject. They find "no relation * * * between the stresses actually observed and the stresses computed by

*This discussion (of the paper by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq., printed in *Proceedings* for April, 1909, but not presented before the Society, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

column formulas," and suggest that the design of details and conditions of manufacture may be such as to make void the usual assumptions entering into these formulas. Mr. Moses.

The whole subject of detailing is connected intimately with that of manufacturing. It is of essential importance economically, but has not received its proper share of attention. At present, working stresses are commonly limited to one-half the elastic limit, principally because of the uncertainty which exists in regard to the proportion of the strength of the main members, which can be developed by the details. With more exact knowledge of the actual distribution of stresses, and with the advance in the art of designing details which will accompany this advance in knowledge, it may become apparent that higher cost of manufacture will be much more than offset by reduced expense for material. Better methods of manufacture will also make possible the use of stronger steels, with all the possibilities such a change foreshadows. The recent rapid increase in the variety of sections rolled shows that this study of detailing need not be confined to the sections which have been standard for the past thirty years.

The authors show by example that a bend of less than $\frac{1}{16}$ in. in the channels of Column No. 1 would account for an increase of stress of 50 per cent. Shop experience shows that, aside from kinks due to handling during shipment from the mills or about the shops, the actual punching of angles, plates, and channels will curve the separate pieces sufficiently to cause such eccentricities as that cited, although the parts are apparently straightened by the lattice. Channels will not only bend, but will also twist, when punched in the flanges. Their flanges will perhaps be kinked at every hole by the blow of the punch. Plates will show buckles between the rivets after driving. Twisters must often be applied to columns during riveting to keep them from assuming a spiral form. Designers insist that lattice bars of columns made of four I-shaped angles, shall intersect on a common rivet, and that the tie-plates shall be of the same thickness as a single bar, expecting the angles to be pulled down by the riveting. They are pulled down, with a resulting kink in the outstanding legs as well as in the riveted leg. Numerous examples of this kind could be cited in support of the authors' statement as to conditions of manufacture.

The authors' experiments on lattice bars show the desirability of more adequate provision for staying the open sides of columns, when such columns are used; and the frequent bending of lattice bars in shipment and erection emphasizes the argument to those acquainted with the facts. A return to closed sections with separate joint pieces is among the possibilities.

The accumulation of further data is to be earnestly desired, and will certainly have a marked effect on the future development of structural work.

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PAPERS AND DISCUSSIONS

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THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.

Discussion.*

By Messrs. W. C. Hammatt, Robert G. Dieck, Charles E. Gregory, E. Kuichling, and Kenneth Allen.

Mr. Hammatt.

W. C. Hammatt, Assoc. M. Am. Soc. C. E. (by letter).—In the writer's opinion, there are few cases when it is advisable to handle storm-water and sewage together; and it is particularly objectionable when the discharge is below water. For a given district, variations of the quantity of sewage proper are slight, while the storm-water will vary from nothing for a great part of the year to a quantity many times in excess of the sewage on some occasions.

In a main emptying below water level, therefore, and designed to carry both storm-water and sewage, the velocity would be so slight when sewage alone was being carried, that sedimentation would take place. Cases are conceivable where the sedimentation would be so great during prolonged dry seasons that the mass would be too great for the first storm to remove, and serious damage might be done to the outfall piping and the lower sewer districts.

It might be claimed that the relief outlets would take care of all the storm-water if the main pipe were obstructed in this way, but if such a thing should happen, and the storm were of a low intensity, the sewage which would take the relief outlet would be diluted so slightly as to be offensive, and as the relief sewers are to empty in exposed places, this would be objectionable.

*This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909, and presented at the meeting of May 19th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

The writer believes that sewage proper and storm-water should be handled in separate conduits and under separate systems. The sewage should be discharged below low-water level in such a manner as to produce proper dilution and removal by currents, as described by Mr. Grunsky. The storm-water should be discharged above tide level, when possible, at the most convenient beach or pier head. The outfall for sewage proper should be of such cross-section that the velocity against the back-water would never fall below 0.5 ft. per sec. Mr. Hammatt.

ROBERT G. DIECK, ASSOC. M. AM. SOC. C. E. (by letter).—In connection with the design of an improved sewerage system for San Francisco, it may be of some interest to state the method of design followed in the storm-water drainage system of Manila, P. I. To understand clearly the essential geographical features of the city a few remarks are necessary. The city is situated at the mouth of the Pasig River, on the east shore of Manila Bay, on an alluvial plain formed of sand and gravel, mingled with loam, brought down from the range of mountains which forms the main drainage slopes of the Island of Luzon. In addition to the major stream, the river has two main branches which take off within the limits of the city, and serve as additional reliefs in times of high tide and floods in the river. One branch extends north and the other south, and both are of about equal importance. There are several smaller streams, which intersect these branches and the main stream, forming a network of canals of more or less importance, which makes the system of waterways practically interdependent. There are also several minor connections to the bay, but these are of lesser importance. Except at extreme low water, practically all the streams are navigable for small river boats, and furnish a ready means of transportation. The larger streams have been dredged where shoaling has occurred. Mr. Dieck.

The ground surface rises from about 2 m. above mean low water in the southern and western portions of the city to about 5 m. above the same datum in the northeastern portion, where small foot-hills are encountered. Practically 75% of the area of the city lies within these elevations. As the average range of the tide does not exceed about 1 m., water is encountered in portions of the town at a depth of 1 m. below the street surface at high tide. Because of this low elevation, the water-table is almost constantly near the surface in large areas of the town; and, for the most part, in the business and residence sections, trenches more than 1.5 m. deep require to be pumped. As these sections required attention at once, it was soon seen that the shallow trenches made necessary by the high water-table would influence seriously the design of the sewers. During the rainy season, which occurs from about June to late in October or even to early December, as a consequence of the torrential rains, the water-table rises, and, in some parts, almost coincides with the street surface, making dry trenching an impossibility without pumping.

Mr. Dieck.

The river has a depth of about 14 ft. at the east end of the harbor (obtained by dredging), decreasing to about 6 ft. at extreme low water at the eastern limits of the city. The main branches have a maximum depth of about 8 ft. Gaugings of these streams and the river have not been made, but, in times of flood, when the current has a velocity of nearly 10 ft. per sec., the discharge of the river must be at least from 50 000 to 60 000 sec-ft. For the most part, the *esteros* (as the branches are called) are sluggish and rather shallow, exposing large portions of their beds at low water. Their frequency, however, makes them admirable recipients of storm-water, and makes further subdivision into drainage districts unnecessary.

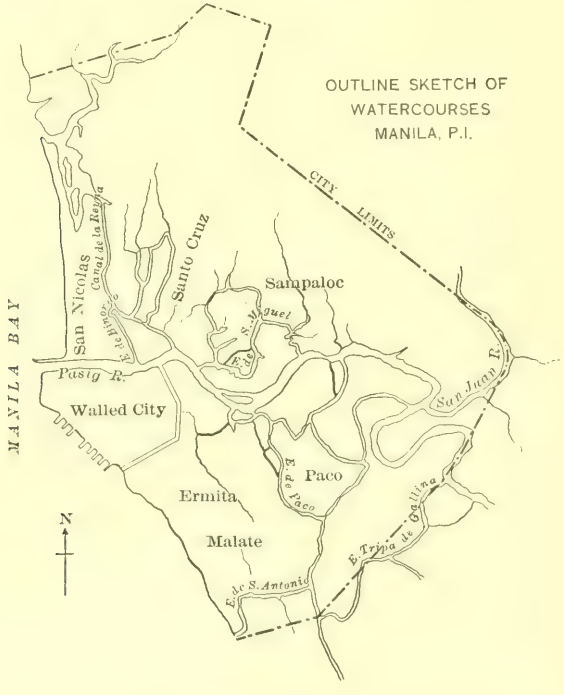


FIG. 12.

Before it was possible to arrive at a satisfactory solution, it was evident that the entire question of street grades would have to be considered, and that some definite plan for these, in opposition to the former, haphazard plan, would have to be adopted. To this end, careful levels were taken throughout the city, and the data were marked on the city plans. Preliminary computations clearly showed that to avoid excessive fill, which, under local conditions, would have proven extremely costly, and to protect property interests against serious

damage following violent changes in grade, a flat gutter grade was absolutely essential. After long discussion, this grade was fixed at the absolute minimum of 0.002. It had been observed that water would stand, on the most carefully constructed work laid to a grade less than this, and as all stagnant water is objectionable in the tropics, it was held that this feature should govern the work, even if in street construction the item of filling were large. As a further limiting factor, it was fixed arbitrarily that the street surface at no point should be less than 0.75 m. above the highest tide. The limits set varied for the various districts. Although it was known that all the low districts would be subject to overflow during high floods in the river, it was still held that some reasonable height of street surface should be fixed in all cases, in order to improve, wherever possible, a naturally bad condition. Mr. Dieck.

Another limitation was in reference to cover. After some observation of 24-in. concrete pipes, under heaviest city traffic, the minimum cover was fixed at 0.60 m. Up to the present this cover has been sufficient. For reinforced sewers no limit was set except an amount sufficient to clear the paving. Under the worst conditions, therefore, with the further provision that the sewers should not be laid below mean low water, because of the likelihood of silting, the sewers had to be designed to fall entirely within from 1.15 to 1.25 m. It was essential, also, that the space occupied by the sewers should be kept at the lowest limit in order to provide for other structures in the narrow streets, these, in some cases, being only 8 m. between building lines, with a clear roadway of only 6 m. At every turn, therefore, certain definite restrictions were to be observed, and also the strictest economy in space and head-room.

A study of the rainfall records was then made. Fortunately, the admirable records of the Philippine Weather Bureau were available, and were examined for maximum intensities for a period of at least 16 years. The records clearly showed that intensities of more than 3.00 in. per hour were not uncommon. Table 20 is a list of typical storms which were considered as probably the worst to be encountered.

Storms having an intensity of 2 in. were common throughout the rainy season, and occurred when the ground was thoroughly saturated, as well as during the drier times. It was evident, therefore, that no relief could be expected from ground storage, but, as the runs in the sewers were short, it was decided that the maximum flow could not occur until about 10 min. after a severe rain had commenced. Table 20 shows that ordinarily the very heavy rainfalls did not last more than from 20 to 25 min., so that if the maximum discharge occurs as stated, and provision is made for lesser rainfalls, there can be flooding only for 10 or 15 min. As there are no cellars or basements in the city, provision against back-water in the sewers was not required, and

Mr. Dieck. it was considered that with safety and at the expense of only a temporary inconvenience, flooding of the roadways would be permissible during unusual storms. Under such cases the sewers would run under a head, which would increase their capacity.

TABLE 20.—TYPICAL STORMS IN THE PHILIPPINE ISLANDS.

Date.	Total rainfall, in inches.	Duration, in minutes.	Excessive rainfall, in inches.	Hourly intensity, in inches per hour.
Apr. 29, 1905	2.56	14	0.79	3.50
May 21, 1892	2.36	30	2.36	4.72
June 8, 1905	2.26	10	0.98	5.90
" 15, 1891	2.16	8	0.59	4.43
Sept 18, 1887	2.06	30	1.57	3.05
Aug. 26, 1903	2.05	30	1.62	3.24
July 19, 1899	2.02	15	Record imperfect.	
Sept. 15, 1891	1.98	12	0.785	3.94
July 16, 1890	1.98		Record imperfect.	
Nov. 16, 1891	1.96		Record imperfect.	
June 1, 1902	1.91		Record imperfect.	
Aug. 6, 1889	1.89	7	0.59	5.51
June 30, 1889	1.86	20	1.18	3.54
Sept. 14, 1898	1.86	17	0.985	3.48
Aug. 27, 1888	1.85		Record imperfect.	
Sept. 17, 1888	1.85	35	1.79	3.07
July 13, 1904	1.81	10	0.73	4.37
Aug. 28, 1897	1.81	16	1.57	5.90
Mar. 9, 1894	1.80		Record imperfect.	
July 12, 1904	1.75	10	0.79	4.72
Sept. 6, 1896	1.73	15	0.985	3.15
July 23, 1888	1.68		Record imperfect.	
Aug. 26, 1886	1.65	15	0.59	2.36
May 29, 1891	1.65	10	0.787	4.72
Sept. 20, 1904	1.62	10	0.71	4.25

It was then decided to design the sewers under the assumption of a 2-in. rainfall, discarding the excessive storms from consideration, relying on the sewers under pressure to discharge the excess, and permitting the flooding of the roadways for 15 min. In the design no account was to be taken of the increased capacity of the sewers due to this head, the excess being viewed merely as a factor of safety in the design. This compromise was made only after the closest preliminary trials and after it was clear that the restricted head-room, flat grades, and the necessary economies in expenditures required some concessions to be made. It was also kept in mind that during severe storms the river and the *esteros* were subject to overflow and that this could not be prevented by any possible provision in the sewers. The results obtained under this assumption, while not satisfactory in all cases, appear to be justified, for no serious flooding had occurred in 1907, two years after the first sewer was built under the general plan.

At first, a run in the gutters of 120 m. had been fixed as the maximum in the built-up sections, but later observations on the gutter flow showed that this was excessive, and the maximum was reduced to 100 m. Sewers with inlets at this distance apart give better satisfac-

tion, and flooding has been lessened. All inlets have direct connections to the sewers, and are provided with silt basins and grating covers. Mr. Dieck.

In the designs, Kutter's formula for discharge, with sewers running full, was used, the values of n being taken as follows:

0.011 for cement plaster finish,
0.013 for clean brick and glazed pipe,
0.015 for local stone.

A velocity of 3 ft. per sec. was sought wherever possible, so as to lessen the likelihood of silting, which had always been the greatest difficulty in the old sewers built under Spanish direction. All drainage areas were kept completely separated, and the connection between the old sewers built by the Spaniards was stopped in order to make the direction of flow definite. To a great extent, the deposits were undoubtedly due to this error in design, and the immediate betterment in the condition of the old sewers following the removal of the connections was made evident by the increased discharge. Wherever possible the old Spanish sewers were utilized, the sides and inverts being made smoother by cement plaster.

In the absence of any data on the character of the run-off, an assumption of 75% was at first made; but this was found to be excessive in large sections and deficient in the closely built-up parts of the Walled City and in the business sections. The final run-off coefficients were fixed as follows:

Heavily built-up	0.90
Ordinary business and residential	0.75
Suburban	0.50

Undoubtedly, these high coefficients lessened the flooding of the streets and are ample for all future work. It is difficult to compare the assumptions with the results obtained by Emil Kuichling, M. Am. Soc. C. E., or with those by Mr. Grunsky, but it is clear that there has been no serious error in the designs.

The *esteros* are in need of improvement, but their betterment would involve the city in such enormous expenditure that no definite plans to this end have yet been evolved. A large amount of dredging is necessary, and the streams must be confined to definite channels by walls. A preliminary section for a light shallow wall of reinforced concrete was designed, but the preliminary estimates of cost were so appalling that this could not be recommended. These *esteros* are the natural reliefs of the river, and their improvement is of the highest importance in the scheme of storm-water drainage. At times the velocity of the current does not exceed 1.0 ft. per sec. and no scour of the bed is possible. A deepening of the streams would result in increased flow, and their present unsanitary condition would be removed. It is quite probable that there must be some control of their

Mr. Dieck. currents, by gates or otherwise, if the reduction of deposits is to be secured, but, at present, there seems to be no necessity to consider that in the design of the confining walls.

From these remarks it will be clear that the drainage problem of Manila has many nice points to recommend it to the attention of hydraulic engineers, and while no claim is made to any originality, it is believed that the solution is somewhat out of the ordinary.

Mr. Gregory. CHARLES E. GREGORY, ASSOC. M. AM. SOC. C. E.—This elaborate paper presents an unusually able theoretical solution of the problem of run-off in storm-water sewers, and is most valuable as an academic discussion of the subject. In the speaker's judgment, however, the author has not given enough weight to the influence of storage on the surface of roofs and streets. After making certain assumptions for a hypothetical case, this part of the problem is very ingenuously and correctly solved for that case. The assumption is made that the rain reaches the sewer from the roofs and streets in maximum volume within 5 min., which is similar to the assumption often made by supposedly careful investigators and writers. The author, however, shows a much better appreciation of what actually occurs than they in that he takes account of the storage on the surface for the time assumed. For many roofs and a few street surfaces this assumption of 5 min. is no doubt approximately true, but a careful consideration of what should occur, theoretically, and a few observations of what actually does occur, will indicate a wide variation from the assumed 5 min., and demonstrate that storage on the surface, particularly for short, intense storms, has a much greater influence on the run-off than is indicated in the paper.

It is not the speaker's purpose to attempt to solve this very interesting and intricate problem completely, but simply to present a few studies of storage on catchment areas, which have been made for the solution of special problems, and to show their relation to the general subject.

Referring to a paper by the speaker and the discussion thereon by A. Marston, M. Am. Soc. C. E.,* it was there brought out that:

"*First.*—The water falling on the so-called 'impervious' area of an ordinary sewer water-shed does not all run off as fast as it falls, but part of it accumulates in increasing quantity on the surface during downpours of moderate length, such as cause the maximum discharges from sewer districts of ordinary size.

"*Second.*—As the storm continues at the same rate, the ratio of run-off from the 'impervious' area to the rate of rainfall increases, owing to the increased depth and velocity of the surface flow toward the sewer, until finally, if the storm lasts long enough, the rate of run-off from the 'impervious' areas becomes equal to 100% of the rate of rainfall."

* "Rainfall, and Run-off in Storm-Water Sewers," *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 498.

Rough observations indicate that the depth of water which accumulates during heavy downpours on an ordinary street, sidewalk or roof, is greater than is generally assumed, and that the mean velocity of flow in the gutters is not as great as it is usually computed to be. Mr. Gregory

The accumulation of rain on a street has been considered theoretically in the following manner:

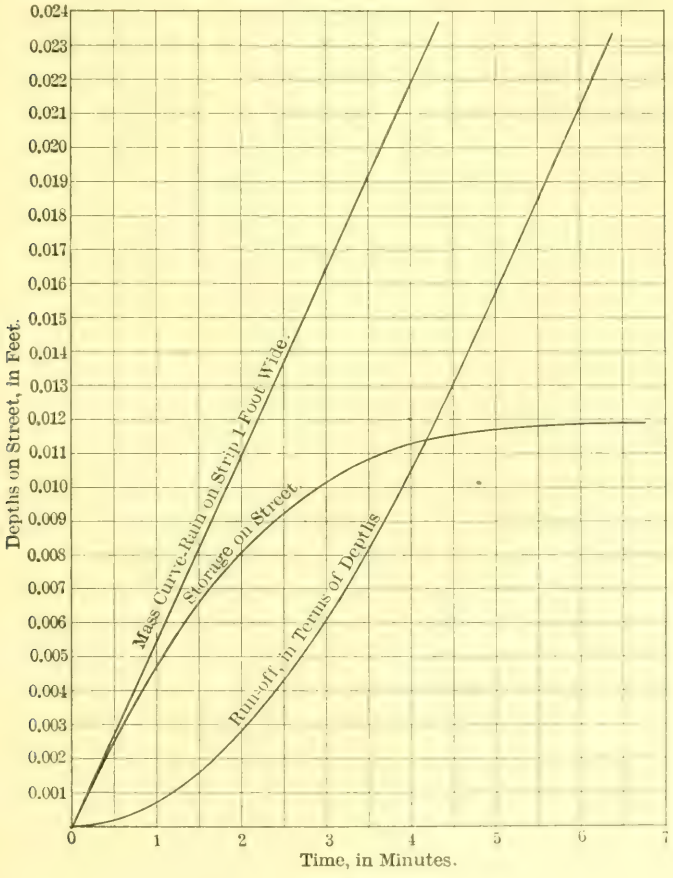


FIG. 13.

The laws and coefficients of flow are not very well determined for very shallow depths, and the impact of the falling rain and other factors materially retard this flow. It was assumed, however, that Kutter's formula was applicable, and, to allow for these retardations, a high value of n was assumed. As an asphalt street, from which the greatest flow may be expected, has a low crown, it was assumed, as

Mr. Gregory. has been observed to be roughly true, that the accumulation of depth would be about uniform over its entire surface. From these assumptions, the diagrams on Fig. 13 were deduced. Three curves are there shown. The mass curve of depths for the entire rainfall is for a rate of 4 in. per hour. Points for the other two curves were obtained from the expression $t = \frac{S(H_2 - H_1)}{I - \frac{Q_2 + Q_1}{2}}$, by plotting values of H_2 and Q_2 in terms of depths.

t = increment of time, in seconds, for surface to rise from H_1 to H_2;
S = area of unit length of half the street = 12 sq. ft.;
H_2 = depth at end of increment of time, t;
H_1 = depth at beginning of increment of time, t;
I = rate of rain, in cubic feet per second, falling on the area, S;
Q_2 = flow, in cubic feet per second, at edge of gutter, due to depth, H_2;
Q_1 = corresponding flow at beginning of time, t, for depth, H_1.

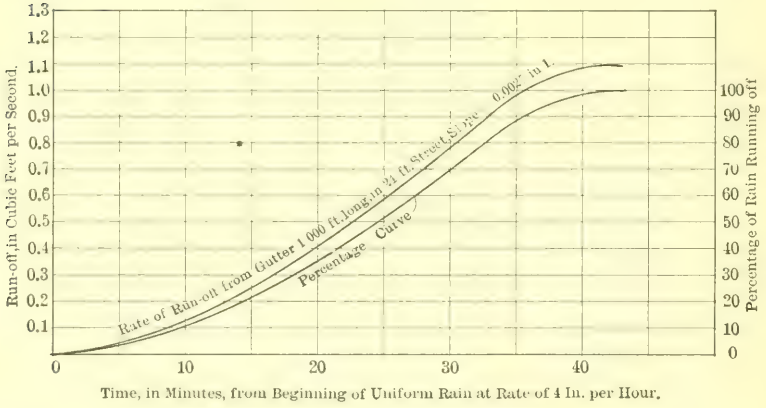

Run-off in Cubic Feet per Second.
Percentage of Rain Running off
Rate of Run-off from Gutter 1 000 ft. long, in 24 ft. Street, S. 25%
Percentage Curve
0.0025 in 1.
Time, in Minutes, from Beginning of Uniform Rain at Rate of 4 In. per Hour.

FIG. 14.

As an extreme example of storage in streets, the following is presented:

The run-off from, and the storage in, the gutter of a 24-ft. street, 1 000 ft. long, with a slope of 0.0025 in 1, is computed in the same general manner as for the street surface. The run-off curve is shown on Fig. 14. For a gutter, or a surface with a uniform slope, the formula is modified to take account of the varying depths, as follows:

$$t = \frac{S(H_2 - H_1) - (A_2 V_2 T_2 - A_1 V_1 T_1)}{I - \frac{(Q_2 + Q_1)}{2}}$$

In this case, t_1, H_2, H_1, Q_2, and Q_1 have the same meaning as before, but are applied to flow in the gutter. Mr. Gregory.

S = average width of each increment of depth multiplied by 1 000 ft.;

I = 1 000 times the inflow from each unit of length, and is taken from the curve of run-off, Fig. 13;

T_2 = total times elapsed from beginning of uniform rain in seconds, or until the depth, H_2, is reached;

A_2 = difference in area between the wet cross-section of gutter for the depth, H_2, and the average wet cross-section for the upper reach of the gutter where the depths are variable;

V_2 = average mean velocity in the gutter until the depth, H_2, is reached;

T_1, A_1, and V_1 bear the same relation to H_1 as T_2, A_2, and V_2 bear to H_2.

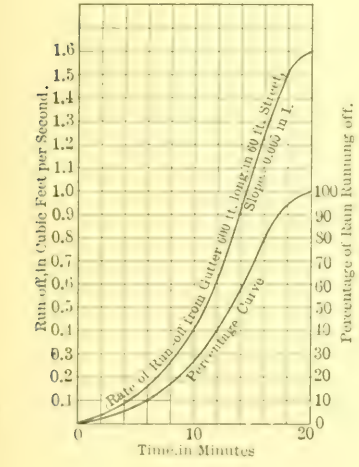

Run-off, in Cubic Feet per Second.
Percentage of Rain Running off.
Time, in Minutes
Rate of Run-off from Gutter 600 ft. long in 60 ft. Street, Slope 0.005 in 1.
Percentage Curve

FIG. 15.

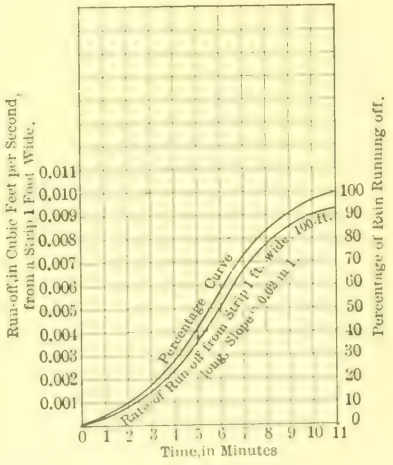

Run-off in Cubic Feet per Second, from a Strip 1 Foot Wide.
Percentage of Rain Running off.
Time, in Minutes
Rate of Run-off from Strip 1 ft. wide, 100 ft. long, Slope 0.02 in 1.
Percentage Curve

FIG. 16.

On Figs. 14, 15, and 16, the rate of the run-off curves is deduced by the same general formula for a rain at the rate of 4 in. per hour, but the storage and inflow curves are omitted, and a curve showing the percentage of rainfall running off is added.

Fig. 15 is for a street 60 ft. wide and 600 ft. long, with an asphalt pavement and a slope of 0.005 in 1.

Fig. 16 is for a roof 25 ft. by 100 ft., sloping at the rate of 0.02 in 1 toward a gutter across the 25-ft. side.

There are a great many roofs and streets comparable to those assumed for the curves on Figs. 15 and 16. A steeper slope, if paved

Mr. Gregory. with stone blocks, would give results equivalent, for flow and storage, to the flatter smooth surface. The Sixth Avenue Sewer District gaugings have been reported by the speaker.* This district contains many 600- and 900-ft. blocks, also many flat roofs, 25 ft. by 100 ft., or larger. It is estimated that it takes 15 min. for water to flow through the sewers from the remote parts of this water-shed to the gauge. In a sewer, the rate of run-off from roofs and streets, as shown in Figs. 15 and 16, may be estimated by assuming that this catchment area is divided into five concentric zones, the outside limits of each being 3 min., 6 min., 9 min., 12 min., and 15 min., respectively, from the gauge. The area of those zones may be taken from the gauge-point outward in the ratio of 1, 2, 3, 4, and 5. The average rate of run-off, expressed as a percentage of the rainfall, for the entire area at the end of 15 min. of uniform rain, at the rate of 4 in. per hour, is found by averaging the percentage of run-off for the average time length of each zone read from the percentage curve on Figs. 15 and 16, respectively.

The averages for the street and roof areas are thus found to be 11% and 48%, respectively. If the catchment area was of such shape that all the zones had equal areas, the percentages would be about 20 and 63, respectively. In a similar manner, a 5-min. time length gives a run-off of about 70% of the rainfall. It is not likely that a rate of 4 in. per hour would be maintained for more than 10 min., and the run-off from the nearby areas would be greatly reduced for the last 5 min. The actual gauged run-off from this area was about 40% for a rain lasting 15 min. This observed percentage may be accounted for by assuming the area to be made up of roofs and streets varying from 5 to 20 min. in time length, taking into account the storage in the catch-basins and sewers, and making allowance for pre-maximum and post-maximum rain rates, the greater influence being exerted by the surface storage. For such rates of rainfall as occur in San Francisco, the velocities of the surface films, due to less depth, would be much less than for a 4-in. rate, and, consequently, the time lengths for similar areas would be greater and have correspondingly greater influence on the run-off. The speaker wishes particularly at this time to call attention to the great influence exerted by the shape, size, character, and slope of each area, and by the frequency of inlets from the streets.

Mr. Kuichling. E. KUICHLING, M. AM. SOC. C. E.—The author deserves the highest credit for having made a more complete analysis of a complicated hydraulic problem than has heretofore been published. The novelty in his procedure is the estimation and consideration of two factors, the first being the quantity of storm-water temporarily in transit on the surface toward the various sewer inlets; while the second is the quantity temporarily stored in the sewers themselves. Both these

* *Transactions*, Am. Soc. C. E., Vol. LVIII.

factors tend to reduce the rate of maximum discharge by making the outflow somewhat less than the inflow until the conduits become entirely filled. Mr. Kuichling

The first factor is obviously dependent on the extent and slope of the territory tributary to each sewer inlet or catch-basin. If the entire area were impervious and divided into a number of equal or equivalent hopper-shaped parts, each having a catch-basin in its middle point, and in which the time required for the water to flow at uniform velocity from the boundary line to the central basin is t_0 minutes, then the quantity of storm-water which will still be on the surface at the end of such time is shown by the author to be two-thirds of the precipitation on the total area during that time.

It should be noted, however, that uniform velocity of flow over the surface of a hopper-shaped area involves a constantly reducing slope for the successive elementary zones, as the depth of the water is constantly increasing from the periphery to the central catch-basin. Such a condition of surface is rarely encountered in municipalities, and hence it must be assumed that the velocity increases steadily as the water passes successively from the larger exterior elementary zones to the smaller interior ones on the same grade. In general, the velocity in a channel, or on a surface, is proportional to the square root of the depth; and as the depth increases very rapidly on the successively reducing elementary zones of like grade, it follows that much less than two-thirds of the precipitation on a uniformly graded component area will still be on the surface in transit to the catch-basin at the end of t_0 minutes.

To illustrate this feature of the case by an example, assume a hopper-shaped area, 1 000 ft. square, having each of the four equal sides graded on a uniform slope to the central catch-basin. Conceive the area divided into five zones, each 100 ft. wide. Beginning at the basin the areas of these zones will be 40 000, 120 000, 200 000, 280 000 and 360 000 sq. ft., respectively, or in the proportion of 1, 3, 5, 7, and 9. Next assume that the rain falls at uniform intensity for a sufficient length of time to allow the run-off from the outermost zone to reach the basin. Now, if it were possible for the velocity to be uniform, as well as the slope, the water flowing over the innermost zone would be about 25 times as deep as on the outermost one. With a uniform grade, however, the velocity will increase from zone to zone, and the depth in the vicinity of the basin will be materially less than the amount just mentioned, since the time of transit is greatly reduced. If the velocity in each zone varies with the square root of the depth of the flow, the time of transit from boundary to center will be less than three times as long as that across the outermost zone, and the quantity of water remaining on the surface at the end of such time will be considerably less than two-thirds of the precipitation during that time.

Mr. Kuichling.

In urban districts, the catch-basins or sewer inlets are generally much less than 1 000 ft. apart, on the average, and the time required for the run-off from the boundaries of the tributary areas to reach the inlets is usually not more than 10 min., as the water soon reaches the gutters, in which the flow is much more rapid than on a plane surface. During this period of time, the precipitation may be 0.5 in. in depth over the entire territory, and perhaps one-half the water may remain on the surface in transit to the inlets, but, at its termination, the surface storage comes to an end if the rain continues as before, and the run-off will thereafter proceed at its full rate to each inlet, since the storage on the surface is not cumulative when the intensity of the rainfall is uniform. For large districts, in which the time of passage through the longest line of sewers is 30 min. or more, during which time a precipitation of 1.5 in. may occur, the retention of a depth of 0.25 in. on the surface during the first 10 min. of the downpour is not of much significance.

The same may also be said with respect to the storage in the network of sewers, as the cubical contents thereof do not often exceed an average rate of 1 000 cu. ft. per acre, including house connections. This capacity is equivalent to a depth of 0.28 in. of rainfall; but, as the whole of the precipitation does not reach the sewers, owing to various causes of loss, it may be assumed that the capacity is equivalent to the run-off from a precipitation of about 0.40 in. In heavy rainfalls of comparatively long duration, however, this storage capacity will usually become filled some time before the end of the storm, and surcharge of the sewers will ensue unless their discharging capacity is adequate.

There can be no doubt as to the propriety of taking these two factors of surface storage and conduit storage into consideration for small districts and short rainfalls, as the aggregate effect may often be very appreciable; but, in the case of large urban districts, having times of transit through the longest line of sewers of more than 30 min., it seems to the speaker that the aforesaid effect will be lost in great degree.

The essential factor in the problem is the relation of the intensity of the precipitation to its duration. Usually, this is estimated more or less arbitrarily from inadequate data, and without an exact knowledge of the extent of territory covered by the rainfall at the recorded intensity. In most cases, also, the extreme observations are disregarded, and the rainfall curve is adapted to the more frequent downpours. If the sewers are proportioned closely to such a curve, there will be no margin of capacity, and it may be questioned whether a moderate reserve capacity, such as would be gained by leaving the above-mentioned two factors out of account, is not a desirable feature. It would certainly be interesting if the author would indicate quantita-

tively how the full consideration of the said two factors would affect the design of a large sewer system in the Eastern States. Mr. Kuichling.

In regard to the rainfall curve that should be used in designing sewers, it has long seemed to the speaker that only those durations should be taken into account for which the intensity or rate per hour was uniform, or nearly so. The object is to ascertain the probable maximum run-off from districts of different magnitude, or duration of flow through the longest line of sewer. Now, it is obvious that the run-off from a homogeneous surface will become largest when the rainfall is greatest and is likewise distributed uniformly over the entire area for a sufficient length of time to allow the water from the outermost boundary to reach the point of observation at the foot of the territory; hence it is of primary importance to deduce from the rainfall data the maximum uniform intensities for different periods of time.

The next step is to determine whether this relation is limited to areas of any particular magnitude, or whether it is possible to obtain a larger rate of run-off at the foot of an extensive area by using precipitations of varying intensity, but having an aggregate duration equal to the time of flow through the longest line of sewer. In general, it will be found that, if the relation between the maximum uniform intensity and its duration has been established correctly, the rate of run-off at the foot of a homogeneous territory of any size will be greatest when the entire area is covered by a rainfall of the highest uniform intensity corresponding to the length of time required for the water to flow to the point of observation from the outermost boundary. Deviations from this rule, however, may possibly occur when the area is not homogeneous, and in such cases one must be governed by the given or assumed conditions of the particular problem.

An instance of non-homogeneous territory is afforded by the following example: Let A equal the number of acres at the foot of a district, with a run-off factor equal to f_1, and the time of flow through the longest line of sewer equal to t_1 minutes; B, f_2 and t_2, C, f_3 and t_3, etc., equal the same elements for the next succeeding component areas; also, let the maximum uniform intensity of the rainfall, in inches per hour (or cubic feet per second per acre), be expressed by $I = \frac{a}{t + b}$, where t is the duration, in minutes. The rates of run-off, in cubic feet per second, from the several component areas, at the end of the time $T = t_1 + t_2 + t_3 + \ldots$ will then be $q_1 = f_1 A I$, $q_2 = f_2 B I$, $q_3 = f_3 C I$, etc., and their sum at the foot of the district will be:

$$Q = (q_1 + q_2 + q_3 + \ldots) = I (f_1 A + f_2 B + f_3 C + \ldots) = \frac{a}{T + b} (f_1 A + f_2 B + f_3 C + \ldots).$$

Mr. Kuichling.

If the district were entirely homogeneous, we would have $f_1 = f_2 = f_3 = \ldots = f$, and hence, also:

$$Q^1 = I f (A + B + C + \ldots) = \frac{a f}{T + b} (A + B + C + \ldots) = \frac{a f N}{T + b},$$

where N denotes the total area of the district $= (A + B + C + \ldots)$.

It will be noticed that if the run-off factor, f, represents the true mean for the whole district, the values of Q and Q^1 will be equal, thus indicating that it is immaterial whether the surface is homogeneous or not.

To pursue the subject a little further, conceive the territory to be homogeneous and divided into equal squares along the sides of which the run-off from each square, or aggregation of squares, moves with equal velocity, v_0, in feet per second; also let the two consecutive sides of each square have an aggregate length of 60 v_0, corresponding to the path traversed by the run-off in 1 min. At the end of T minutes, the number of squares contributing to the discharge at the lower corner of the first square will be $T \frac{(T+1)}{2}$; and, as the area of each square is $m = (30\ v_0)^2$ square feet, the total tributary area, in acres, will be $N = \frac{T(T+1)\,m}{2.43560}$, with $Q = I f N = \frac{a f m}{87\,120} \times \frac{T(T+1)}{T+b} = c \times \frac{T(T+1)}{T+b}$.

We thus have Q as a simple function of T, the form of which shows that Q increases directly with T and attains its greatest value only when T is greatest, or when the entire area contributes to the discharge. This result is due to the peculiar form of the expression for the factor, I. With some other form for I it may happen that Q will become a maximum for a value of T less than that corresponding to the time for flowing through the longest line of sewer in the district.

From this brief analysis of the run-off problem, it appears that attention should be directed mainly to the proper form of expressing the maximum uniform intensity of the rainfall, I, in terms of its duration, t, in minutes. By the use of this factor, the indefinite question of seeking a maximum, Q, with rainfalls of variable intensity during the total time, $T = (t_1 + t_2 + t_3 + \ldots)$, will be entirely avoided, as the former necessarily embraces all combinations of the latter.

The next matter of importance is the investigation of the proper value of the run-off factor, f; but, as the author has not given the results of his own experiments, there is no reason for discussing it here at considerable length. Few direct and reliable measurements of the run-off from a given territory and uniformly distributed rainfall appear to be available. In some cases the rainfall is observed at only a single point in or near the district, and no proof of uniform distribution is offered, while, in other cases, both mean intensity and

distribution are estimated. It is very desirable, therefore, to increase the number of accurate measurements for surfaces of different character, and to include therein the actual time of passage through the longest line of sewer, as well as the time required for the water flow over the surface from the outermost boundary to the nearest catch-basin or inlet. Mr. Kuichling.

To exhibit the range in the estimated values of the run-off factor, f, or the ratio of the run-off to the rainfall at the time of maximum discharge, the following data are submitted:

For roof surfaces assumed to be water-tight... f =	0.70 to 0.95
" asphalt pavements in good order.....	0.85 to 0.90
" stone, brick, and wooden block pavements with tightly cemented joints.....	0.75 to 0.85
" same with open or uncemented joints....	0.50 to 0.70
" inferior block pavements with uncemented joints	0.40 to 0.50
" macadamized roadways.....	0.25 to 0.60
" gravel roadways and walks.....	0.15 to 0.30
" unpaved surfaces, railroad yards and vacant lots.....	0.10 to 0.30
" parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil.....	0.05 to 0.25
" wooded areas or forest land, depending on surface slope and character of subsoil..	0.01 to 0.20

Others do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with general average values of f, as follows:

For the most densely built-up portion..... f =	0.70 to 0.90
" the adjoining well built-up portion.....	0.50 to 0.70
" the next adjoining residential portions, with detached houses and buildings.....	0.25 to 0.50
" the suburban portions, with few buildings.	0.10 to 0.25

Similar variations are found in the estimates of maximum intensity of rainfall for which provision is to be made in the sewers or other drainage channels, even in neighboring localities where the rainfall has the same character and average annual depth. In view of such large differences in practice in the choice of f and I, there is little encouragement for the adoption of refinements in the general theory of the subject.

KENNETH ALLEN, M. AM. SOC. C. E.—It is a good thing to place on record the conditions governing the design, and a description of the essential features, of an important improvement of this kind. Mr. Allen.

Of the fifteen cities of the United States having in 1906 a popula-

Mr. Allen.

tion of more than 300 000, all are situated near bodies of water of such size that up to the present their sewage is discharged into them in a crude condition. Baltimore, however, in order to protect the oysters of Chesapeake Bay, is constructing a purification plant, and in order to avoid an undue pollution of the water, Chicago, San Francisco, Boston, and Washington have spent large sums to secure more favorable points of outfall. Without question, in view of increasing populations and a growing sensibility to cleanliness and sanitation, the next two decades will see extensive improvements in most of the other cities in the list.

Of these fifteen cities, four are on the Great Lakes, five (including Chicago) drain to the Mississippi or Ohio Rivers, and six deliver their sewage almost directly to the ocean. San Francisco, however, is especially favored in being able to locate her outlets—ultimately, at least—in strong currents of deep water which trend off shore and soon pass out to sea. New York and Boston are also located on great tidal bays, but the waters of Upper New York Bay oscillate between the Hudson and East Rivers and the Narrows in close proximity to a dense population for several days before being finally dispersed in the Lower Bay or ocean, while Massachusetts Bay is studded with islands, and the shores are lined with towns and resorts, so that great care has been required in selecting points of outfall to avoid offense.

With a discharge into large bodies of water there should be sufficient dilution to prevent putrefactive conditions, and the problem resolves itself into one of rapid dispersion. This can be secured with a single outlet where the current is swift in comparison with the volume of sewage. At San Francisco, as at Boston, interception and a discharge at a few well selected points, after the removal of the grit and coarser floating matters, has been decided on as the best plan. To secure a more rapid dispersion, multiple outlets are provided in the Elbe at Hamburg, and have been proposed for the Passaic Valley Sewer in New York Harbor; but where cities are located on sluggish streams, it is sometimes preferable to maintain numerous independent outlets in order to afford opportunity for a more rapid mixing with the water. The question of interception to one or a few outlets, as at Buffalo or Louisville, or a direct discharge at numerous points, as at New York and Philadelphia, is a local one, and must depend on the circumstances; but it may be said that the tendency is toward the former plan for large cities, so that it has generally come to be looked on as the proper procedure.

The question as to the value of float experiments in locating sewer outfalls is sometimes raised. The speaker believes them to be of great value as indicating the direction in which sewage may be expected to flow under similar conditions. It is true that a surface float furnishes no measure of bottom velocities, which may be quite contrary in

direction; that the course taken by one float may be quite different from that of one set adrift a few minutes later, although under conditions which appear to be identical; and that it is influenced more or less by the wind; but in answer it may be said: Mr. Allen.

First.—A surface float represents the motion of that portion of the current which comprises the greater part of the sewage, through the tendency of the latter to rise and spread out over the surface, diffusion in a vertical direction being comparatively slow; and if bottom or mean velocities are desired these may be determined, approximately, by the use of double or rod floats.

Second.—Although the trace of a single float will not represent a true average result, yet it will show the path that sewage discharged at the starting point might take; and from a sufficient number of records the probability of its going in any given direction may be predicted with considerable certainty.

Third.—A float is influenced by the wind; but this, with properly designed floats, is mainly through surface currents induced by the wind, and it is the trend of the surface currents which determines the path that sewage would follow. The float, therefore, gives the desired information.

In questions involving dilution, depending on the net discharge of tidal and upland waters, the speaker does not consider results from floats as reliable as those obtained by current meters, as it is difficult, by means of the former, to integrate the motion of the filaments properly in a vertical section. Even with the current meter it is a long and expensive operation, if reliable results are to be obtained on tidal streams. An ingenious apparatus for facilitating such work was devised some years ago by Mr. Luigi d'Auria, and used by him on some public work undertaken under the direction of the late William Ludlow, M. Am. Soc. C. E. This device consisted of a pole which could be lowered vertically to the bottom of the stream from the side of a pontoon. An endless cord which passed through pulleys near the top and bottom of the pole was used to carry a ball of half the specific gravity of water to the bottom. At a given signal the ball was released, and, rising to the surface at a speed which a computation showed would be uniform, and then being caught by a floating grillage, the mean velocity was obtained by dividing the measured horizontal distance traversed by the time taken to rise to the surface.

Continuous records of single floats covering several tides are valuable in determining the range of tidal oscillation and the resultant movement seaward, but, to secure information regarding the probable dispersion of sewage, the plan used by the speaker, on Chesapeake Bay, of setting a large number of floats adrift at short intervals and locating them as frequently as practicable—each being identified by a number—is an excellent one to adopt, being rapid and economical.

Mr. Allen. The importance of a careful consideration of local rainfall records is well illustrated in the case of San Francisco, where, if Eastern practice had been blindly followed, the size and cost of the sewers would have been greatly increased. In this respect it would be interesting to learn why the particular form of rainfall formula was adopted, and whether the more usual and simpler form, $I = \frac{a}{t+b}$, would not have answered equally well.

In forecasting the flow of sewage from a given district, the infiltration of ground-water is often an important but uncertain factor. The daily volume has been stated variously in gallons per mile of sewer, per acre or square mile drained, per capita of population, per linear foot of sewer joint, and per square foot of surface. The rational way is to consider infiltration as a function of the sewer surface, if of continuous concrete or brick, and of the linear foot of joint in the case of pipe or other sectional forms of construction.

The perviousness of the soil affects the rate of leakage to sewers. In New Orleans, George G. Earl, M. Am. Soc. C. E., Superintendent of the Sewerage and Water Board, found that:

"In open and unpaved areas we are probably getting double the amount estimated at times of very heavy rains, particularly when they follow a long dry spell, which has cracked open our clay soil, with deep and frequent fissures.

"Whether this flow is all seepage or partly due to flooded surfaces where manholes exist and where, even, some one may open a manhole cover occasionally, we cannot tell as yet * * * *."

The allowance for infiltration was 0.003 sec-ft. per acre or 31 800 gal. per mile per day, and an estimate made after construction showed this figure—which is the maximum used for San Francisco—to be substantially correct.

In studies made for the sewerage of Baltimore, in 1897, Messrs. Hering and Gray, the Consulting Engineers for that work, recommended an allowance ranging from 0.00024 to 0.0018 sec-ft. per acre.

J. N. Hazlehurst, M. Am. Soc. C. E., secured a somewhat exceptional result* with an 8-in. vitrified pipe laid in 2½-ft. lengths, with a mortar joint of 1 part Louisville cement and 2 parts sand. This was kept covered for 6 hours with a damp cloth, and then immersed for 24 hours. The infiltration was then found to be only 1 329 gal. per mile per day.

X. H. Goodnough, M. Am. Soc. C. E., reported† that on 137 miles of pipe sewers in Massachusetts, varying from 8 to 30 in. in diameter, the infiltration was about 40 000 gal. per mile per day before connections were made, and that it amounted to 70 000 gal. per mile per

* *Engineering News*, August 27th, 1903.

† *Engineering News*, August 27th, 1903.

day on 700 miles of sewer, but that 80 000 gal. per day is estimated in the extensions of the sewers of the Metropolitan District of Boston. Mr. Allen.

In Malden, Mass., the infiltration was 50 000 gal. per mile per day, 70 gal. per capita per day, or 0.015 sec-ft. per acre.

D. H. Maury, Jr., M. Am. Soc. C. E., mentions experiments made in Peoria, Ill., on a 24-in. vitrified conduit laid in 2-ft. lengths, with 1:1 Portland cement mortar joints, in which the infiltration found by numerous measurements was generally from 8 000 to 35 000 gal. per mile per day, although, in some instances, the figure exceeded 60 000 gal.

The amount of infiltration, of course, will depend largely on the ground-water level, 30 000 to 50 000 gal. per day for each mile of well-laid pipe sewer being probably a fair allowance under ordinary circumstances, but where the exclusion of ground-water is important, this may be reduced by special precautions, and where the sewer lies above the ground-water level there will be a leakage from instead of into the sewer.

Alexander Potter, Assoc. M. Am. Soc. C. E., has secured good results by making joints with sulphur, sand, and pitch, and the speaker has done this by the use of tar pitch and cement, kneaded by hand to the consistency of dough, as has been the practice for a number of years, on the construction of sewers in Atlantic City. Some 20 years ago the speaker attempted to secure a water-tight joint with natural (Black Diamond) cement under furnished specifications, but he was unsuccessful. More recently, however, he has laid about ½ mile of 6-in. and 8-in. pipe, from 0 to 9 ft. below ground-water level, under the usual specifications of Portland cement mortar with a yarn gasket, which, after repairing about a dozen small leaks, the contractor made practically water-tight. To secure impervious work in this way, however, is more difficult and costly than it should be, and the speaker believes that some bituminous joint, poured like a lead joint, may eventually supersede cement—as it already has done to a certain extent abroad—and will furnish a more elastic joint, and, at the same time, a more economical result.

In the case of brick sewers, infiltration varies greatly with the quality of the work and the depth below ground-water level, but there are few definite data to serve as a guide.

Owing to the general substitution of concrete for brick in larger sewers, there is less infiltration than formerly, and this may be done away with practically by careful grading of the aggregate, wet mixtures, and continuous work. In the speaker's experience the greatest difficulty is at the springing line, where the joint between arch and invert, in spite of the bonding key, offers the least resistance to the entrance of external water under pressure. Infiltration in concrete sewers has been prevented entirely by water-proofing laid between the

Mr. Allen. concrete and a brick lining, but the extra cost of the water-proofing and the lining is hardly warranted in order to prevent the comparatively small amount of leakage which would otherwise occur. In a reinforced concrete sewer built at Waterbury, Conn., the infiltration under a head varying from 4 to 12 ft. was found to be 2.08 gal. per day per square foot of surface.

The poor alignment, size, and shape mentioned by the author, as existing in the old San Francisco sewers, have been similarly noted in Boston, New York, and Baltimore, and may be found in most cities large enough to have had sewers more than 50 years old. As a rule, smaller municipalities are freer from such absurdities; but, as an adopted standard design, the old San Francisco 3- by 5-ft. intersection, constructed uniformly 10 ft. below the surface, irrespective of topography or connecting grades, is unique.

The author alludes to changes in the elevation of bench-marks. This, to the extent of 0.10 ft. or more, may be due to upheaval by frost, or by a lowering of the ground-water level on account of the construction of drains, the sinking of wells, etc., depending on the character of the soil, unless the foundation extends to bed-rock. Aside from this, variations have been known to occur, due to more general movement of the earth's crust. Large areas are known to be involved in these movements, such as the gradual sinking of the New England Coast, which, as John R. Freeman, M. Am. Soc. C. E., has pointed out,* is taking place at the rate of about a foot a century.

But there is reason to believe that there may be other alterations in the relative elevations of points within such short distances of each other as to affect the bench-marks of a municipality.

Some time ago, Horace Andrews, M. Am. Soc. C. E., brought a case in point to the attention of the speaker in connection with the "re-levelling of a number of points of the Bavarian precise levels which were originally determined in the interval between 1869 and 1873 and the mean levels taken in 1897."† These were both levels of the highest order, with refinements as to accuracy of standard and permanent marking of bench-marks, which, unfortunately, are often disregarded.

"The discrepancies revealed were about equally plus and minus and in only one or two cases did there seem to have been some gross change in the foundation of the building on which the mark was placed.

"The differences in this Bavarian work were found by touching on nearly fifty points scattered over a large region. * * * Discrepancies of from 13 to 14 mm. seemed to be common, while several were as large as 15 to 17 mm. and two were of 41.5 mm. and one was 86.7 mm. In addition to these was the gross change alluded to of 233.5 mm. in one point."

* Report on the Charles River Dam, 1905.

† *Zeitschrift für Vermessungswesen*, August 1st, 1900.

Mr. Allen.

About 1894, precise levels were run in connection with a general topographical survey then being made of Baltimore by Colonel H. T. Douglas, and numerous bench-marks were established. Twelve years later the speaker had occasion to test a good many of these, and, although precise levels were not run, the discrepancies noted were checked so carefully that he was convinced that numerous changes had occurred during the intervening years. This opinion was confirmed by results reached on another division of the same work, where it was observed that the greatest discrepancies occurred where the slopes were most abrupt. Discrepancies of this kind might appear to be due to a difference in the graduation of the rods, but as these had been compared carefully with a standard, the conclusion reached was that this was not the cause. The apparent errors of elevation noted varied up to several hundredths of a foot.

It would be interesting to learn the general experience of municipal engineers who have had the opportunity of checking benches established by precise methods after the lapse of several years.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE ADGATE, M. Am. Soc. C. E.*

DIED JANUARY 23D, 1909.

George Adgate was born on September 28th, 1845, at Keeseville, New York, and was the eldest of a large family of children. His ancestors emigrated from England to Massachusetts and Connecticut during the earliest Colonial times. Through a marriage between the son of the first Adgate who came to this country, and a granddaughter of Elder William Brewster, the Adgates were placed in the line of descent from the Pilgrims.

The family has been useful and influential in public affairs throughout its history. Mr. Adgate's great-grandfather, Judge Matthew Adgate, was a member of the New York Legislature and of the convention which formulated the first constitution of the State of New York. His grandfather and father were also members of the New York Legislature.

After his early education at Keeseville Academy, one of the best of the old-time schools, Mr. Adgate spent two years keeping a store in the oil regions of Pennsylvania. Later he became connected in business with his father-in-law, Mr. Whitney, a contractor on Lake Champlain. The work consisted principally of dock-building, much heavy construction of that kind being necessary in those days, before any railroads had been built on the shores of the lake. The fifteen years he spent in struggling with the difficulties of subaqueous work, often under severe climatic conditions, were instrumental in developing his judgment and resourcefulness.

In 1882 Mr. Adgate was engaged by William Sooysmith and Son as Manager of bridge foundation construction, most of which involved the use of pneumatic caissons. He was engaged with this firm and its successors throughout the remainder of his busy life. Here his experience and sound judgment were often tested while building foundations in large rivers where varying conditions as of floods and ice gorges had to be anticipated and provided for in advance. In this warfare with the elements it can be truthfully said that he never lost a battle.

He was in active and responsible charge of the building of the foundations and piers of the following bridges:

* Memoir prepared by E. L. Abbott and Daniel E. Moran, Members, Am. Soc. C. E., and Edwin S. Jarrett, Assoc. M. Am. Soc. C. E.

Terre Haute, Ind. "Big Four" Bridge over the Wabash River.
Van Buren, Ark. Bridge over the Arkansas River.
Needles, Cal. Bridge over the Colorado River.
Cuivre Junction, Mo. Bridge over the Cuivre River.
Boone, Iowa. Chicago and North-Western Bridge over the Des Moines River.
Harvey, Iowa. Chicago, Rock Island and Pacific Bridge over the Des Moines River.
Kansas City. Chicago, Rock Island and Pacific Bridge over the Kaw River.
Clinton, Iowa. Chicago and North-Western Bridge over the Mississippi River.
Ottawa, Can. Interprovincial Bridge over the Ottawa River.
Glasgow, Mo. Bridge over the Missouri River.
East Omaha, Nebr. Bridge over the Missouri River.
Plattsmouth, Nebr. Chicago, Burlington and Quincy Bridge over the Missouri River.
Sibley, Mo. Bridge over the Missouri River.
Sioux City, Iowa. (Pacific Short Line) Bridge over the Missouri River.
Arthur City, Tex. Bridge over the Red River.
Gainesville, Tex. Bridge over the Red River.
Fort Worth, Tex. Gulf, Colorado and Santa Fé Bridge over the Trinity River.
Marietta, Ohio. Bridge over the Ohio River.

He was also in charge of many smaller bridges.

Mr. Adgate was associated in the construction of the foundations of the Baltimore and Ohio Bridge across the Susquehanna River, at Havre de Grace; he also supervised the sinking of several deep shafts and the building of numerous docks, dams, intake piers, lighthouses, and other foundation work. At the time of his death, January 23d, 1909, he was Western Manager for The Foundation Company of New York City.

Mr. Adgate inherited a wholesome and rugged constitution, which enabled him to contend successfully with the many adverse conditions and surroundings incident to his work in the difficult kinds of construction on which he was continuously engaged.

His organizing ability was wonderful, and his keen foresight, calm judgment, and iron determination were chiefly instrumental in the success of the many works for which he was responsible.

Personally, Mr. Adgate was kind, just, and lovable. He could direct the operations of others so as to obtain their best efforts and loyal support, and the engineers and associates with whom he worked, had, to the last, the highest regard for his ability and integrity. He

had a fund of quiet humor, which came to the aid of his associates and helpers in the most trying situations and lightened their load. He was always ready to help young men after they had shown their worth, but believed in trying them out in a rough but kindly way. He was genial and bluff, but on questions of principle stood like a rock against persuasion or threat.

He was married on September 19th, 1866, to Miss Martha Whitney, who survives him, together with a daughter, Mrs. J. R. Dean, of New Britain, Connecticut, and a son, Frederick W. Adgate, Assoc. M. Am. Soc. C. E., of Chicago.

Mr. Adgate was elected a Member of the American Society of Civil Engineers on April 1st, 1896.

WILLIAM PRICE CRAIGHILL

WILLIAM PRICE CRAIGHILL, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1909.

A long and active life spent in the service of the country builds its own enduring monuments. Little remains to his friends other than to pen a few inscriptions, which, though inadequate, may serve to remind the passer-by more clearly of the man whose work is there in testimony of the duty done and the good accomplished. An engineer, eminent in his profession, a Past-President of the American Society of Civil Engineers, and for a time the head of his Corps in the Army, must necessarily have such monuments widely scattered over our land. It is so with General Craighill. The records of his work are found in our improved rivers and harbors, in our fortifications, and also in the lives of the younger men of the profession who were associated with him and whom it was his delight to help.

William Price Craighill was born in Charlestown, West Virginia (then Virginia), on July 1st, 1833, the eldest son of William Nathaniel and Mary Elizabeth (Brown) Craighill. He was of mixed English and Scotch blood, and his ancestors bore a prominent part in the early history of Virginia. The earliest of the American Craighills, William, is believed to have been born near Carlisle, England. His son, Nathaniel, served in Captain William Washington's company of Minute Men, in 1774-75, and later was in the Continental service. The first William Price Craighill, son of Nathaniel, was born at Craighill Point, Westmoreland County, Virginia, served as a volunteer in the War of 1812, and was a member of the Legislature of Virginia about 1817. General Craighill, his grandson, was named after him. Among other ancestors can be named the Littles, one of whom was a friend and neighbor of General Washington at Mount Vernon, and one of his pall-bearers; the Browns, who claim descent from one of the founders of Jamestown; and the Rutherfords, of an ancient Scottish family, one of whom, Robert, the maternal great-grandfather of General Craighill, was also an associate and friend of Washington, as a surveyor and fellow-officer of the early Indian wars, and a participator in the political agitations in Virginia immediately preceding the Revolution. He held many places of trust in Virginia, and was the first Member of Congress elected from the west of the Blue Ridge, having represented Berkeley County from 1793 to 1797.

Among the founders of the Charlestown Academy, in 1795, were three of the grandfathers of General Craighill, Little, Craighill, and Rutherford. Here the General received his early schooling, following his grandfather and father, and here his sons succeeded him. General

* Memoir prepared by the following Committee: General Charles W. Raymond, Colonel William M. Black, Lieutenant-Colonel Gustave J. Fiebeger, Bernard R. Green and John Thomson, Members, Am. Soc. C. E.

Craighill was for years a Trustee of the school, and finally President of the Board. On July 1st, 1849, at the age of sixteen, he became a Cadet at the United States Military Academy, the youngest of his class, the roll of which contained many names destined for fame, and among these may be mentioned McPherson, Schofield, Sheridan, Sill, Sooy-Smith, Terrill, Vincent, Tyler, Sweitzer, and Hood. He was graduated in 1853, second in the class, which McPherson led, and was commissioned Brevet Second Lieutenant, Corps of Engineers. His subsequent advances in rank in the Corps of Engineers of the Army were as follows: March 3d, 1855, Second Lieutenant; July 1st, 1859, First Lieutenant; March 3d, 1863, Captain; November 23d, 1865, Major; January 2d, 1881, Lieutenant-Colonel; January 10th, 1887, Colonel; May 10th, 1895, Brigadier-General and Chief of Engineers. On March 3d, 1865, he was made Brevet Lieutenant-Colonel "for faithful and meritorious services during the war, and particularly for services in the defence of Cumberland Gap, and the ulterior operations of General Morgan's forces." He also received the brevet of Colonel "for gallant and meritorious services during the rebellion," but this he declined.

When the Legislature of Virginia adopted the ordinance of secession, in 1861, Lieutenant Craighill was confronted by the grave question whether he should follow the fortunes of his native State or remain in the service of the Union. On the one hand were all the associations of his youth, the pride of his ancestry, and the certainty of immediate professional advancement; on the other hand, a solemn sense of duty and the sincere belief that the ultimate welfare of the whole country depended on the success of the cause of the Union. After the most anxious and thoughtful consideration, he decided that it was his duty to remain in the service of the Union; but he refused to seek advancement in a war in which he found himself opposed to his own people, and thus voluntarily surrendered the opportunity of obtaining the high honors and personal distinctions which were attained by so many of his associates of similar age, training, and experience in both the Confederate and the Union armies.

From 1853 to 1856, Lieut. Craighill was on duty on the improvement of the Savannah River and in the construction of fortifications at its mouth, on fortification work at Dry Tortugas, and in fortification construction and harbor improvements in Charleston Harbor. He was stationed at Fort Sumter during the yellow fever epidemic which ravaged Charleston in 1854-55, and while there became cognizant of one of the terrible abuses which slavery made possible, which made on his mind an indelible impression of the evils of the system. In 1856, he went to Washington as Assistant to the Chief of Engineers, General Totten, Hon. M. Am. Soc. C. E. In later life, he was fond of recalling the methods of that talented and painstaking official, who, with his own

hands, frequently made working drawings of the details of the beautiful stonework to be found in the fortifications of that date, showing a possibility for the careful individual study of the problems of the office, which must be a cause of envy for General Totten's overworked successors of later days. In 1859, Lieut. Craighill was ordered to the Military Academy as Principal Assistant Professor of Engineering, and remained on that duty until June, 1863. While at the Academy, he served for two years as Treasurer, and for one year was in command of the detachment of Engineer soldiers at West Point. He also compiled "The Army Officer's Pocket Companion," a treatise which proved very useful to the Volunteer Army, and translated Dufour's "Cours de Tactique" and, jointly with Captain Mendell, Jomini's "Précis de l'Art de la Guerre."

In the summer of 1862 he was Chief Engineer of the Division of General George W. Morgan, Army of the Ohio, and was engaged in the defense of Cumberland Gap. In his report on the campaign, dated October 12th, 1862, General Morgan writes:

"Nor can I close this report without calling the attention of the Commanding General to the important services rendered me by Lieut. W. P. Craighill of the Engineer Corps. He is an officer of distinguished merit, and is thoroughly informed on all subjects connected with the art of war. He would make an able Chief of Staff, or fill with high credit any other position to which he may be assigned, and deserves a much higher grade than he now holds."

In the summer of 1863, Captain Craighill was Chief Engineer of the Department of the Monongahela, and was engaged in the construction of the defences of Pittsburg, then threatened by the raid of General John Morgan. Later, he was Assistant Engineer in the construction of the defences of Baltimore, and in 1864, was Chief Engineer of the Middle Department and Eighth Army Corps. In 1864, he again went to the Military Academy, where he served as Assistant Professor of Engineering, Instructor of Practical Military Engineering, in command of the Engineer Detachment, and as Treasurer, serving also on Boards for the defences of San Francisco, for fortifications at Willets Point, and for New York.

In November, 1865, he was sent to the Baltimore District, in charge of the construction of the fortifications and of river and harbor improvements, and, with the exception of the tour of duty from 1866 to 1870 as Assistant to the Chief of Engineers, General A. A. Humphreys, Hon. M. Am. Soc. C. E., he remained on that duty until promoted to be Chief of Engineers in 1895. His fortification work included the construction of works at Baltimore, Washington, and Hampton Roads. The river and harbor works extended from the Susquehanna to the Cape Fear, and as far west as the Great Kanawha, in West Virginia. In Baltimore Harbor, the dredged channel constructed by him bears his name.

The improvement of the Cape Fear River presented a notable engineering problem, and the successful closing of the false mouth across the shifting sands of the coast was a work of great importance. In this work was evolved the log and brush mattress, used extensively later on the South Atlantic Coast. Perhaps the most notable and successful of the works was the canalization of the Great Kanawha River, in which the Chanoine system of movable dams is used. It is believed that the introduction of this system in the United States is mainly due to the late Colonel Merrill, with whom Colonel Craighill visited Europe in 1878, for an inspection of works of that character; but the first Chanoine dams actually constructed and placed in service in the United States were on the Great Kanawha River, under Colonel Craighill.

In addition to these regular duties, Colonel Craighill served on many boards formed for the consideration of projected improvements, among which may be named those for the Columbia, Mississippi, Hudson, Ohio, Delaware, Potomac, Cape Fear, Great Kanawha, James, and other rivers, and the breakwaters for Delaware Bay, Sandy Bay, Massachusetts, and San Pedro, California. For several years he was a member of the Light-House Board and of the Board of Engineers in New York, to which were referred questions relating to fortifications and river and harbor improvements.

In 1884 the system of grouping a number of river and harbor districts into a division was inaugurated. From that date until 1895 Colonel Craighill was the Division Engineer of the Southeast Division. When this division was first formed, it had several of the younger officers of the Corps of Engineers as its district engineers, and was known as "the kindergarten." The Division Engineer's visits were always welcome, and he took a keen interest in the work of the younger men, approving their methods whenever practicable. "For," he said, "I have found that generally there are several ways of accomplishing a given result, and that it is best to follow the plan of the man who is to do the work providing that plan be sensible"—a truth not always appreciated by superiors. At another time, he said to a young officer who was reporting to him for duty, "Mr. ———, I propose to be the laziest man in this district, and do not propose to do anything that I can have done by one of my assistants." It is needless to say that, under such a man, the assistants always did their best.

As Chief of Engineers, it fell to General Craighill to arrange for the prompt construction of the many new works of defence for which Congress had for the first time for years made large appropriations. Under him, a number of changes were made in the methods and policy of the Engineer Department, the most important of which was the inauguration of the practice of having the type plans of the batteries, with the general project for the defense of a harbor, referred to the

local engineer, prior to final adoption, in order to obtain the benefit of his local knowledge and experience. Criticisms of the plans were requested from the various branches of the service interested. The results were immediate and beneficial. Due to his insistence, the development of the disappearing carriage for 12-in. guns was hastened by the Ordnance Department, thus permitting the abolition of the very expensive lift type of battery, which, before that, was the only type for 12-in. guns affording protection while they were being loaded. He also changed the type of the mortar batteries, and had important improvements made in the submarine mine system. Under him, was begun the practice of disseminating important technical information among the officers of the Corps by means of mimeographs.

Incident to his service, General Craighill had to appear frequently before the various committees of Congress, where he made many friends. His succinct and straightforward answers to questions were always received with marked attention. His sound judgment and evident thorough knowledge of the subject at issue caused his opinions to have great weight. An evidence of this is found in the history of the Washington Aqueduct tunnel. This had been abandoned after the discovery of fraudulent work on the part of the contractors, and the tunnel project had become thoroughly discredited in Congress. After an expert examination into its merits, General Craighill presented to Congress a full and convincing report, and, as a result, the necessary appropriations were made and the work was successfully carried to completion. The tunnel to-day forms an important link in the Washington water-supply system.

On February 1st, 1897, General Craighill was retired from active service at his own request. Excepting a limited service as a member of the Board of Consulting Engineers of the Dock Department of the City of New York, his retirement ended his active career as an engineer. He died at Charlestown, West Virginia, on January 18th, 1909. The immediate cause of his death was stated by his physicians to have been "disability following a stroke of apoplexy causing paralysis of the right side."

General Craighill was married on October 14th, 1856, to Mary A. Morsell, daughter of Judge James S. Morsell, of Washington, D. C., and, after her death, was married, on September 22d, 1874, to Rebecca Churchill, daughter of the Reverend Alexander Jones, of Virginia. He had three sons and three daughters. One of the sons, William E. Craighill, was graduated second in the class of 1885, United States Military Academy, and is now a Major of the Corps of Engineers, U. S. Army.

Of civic duties General Craighill did his rightful share. The Episcopal Church, of which he was a life-long member, showed its appreciation by making him its deputy from the Diocese of West

Virginia to ten successive Triennial General Conventions, beginning with that of 1880, and also a delegate to the Pan-Anglican Conference in London in 1908. He was a member of the Malta Lodge of Masons, Charlestown, West Virginia, from July, 1855, to his death. In 1888 he became a member of the American Association for the Advancement of Science. He was, in 1895, the sole honorary member of the Historical Society of Maryland. In 1897 the Washington and Lee University conferred on him the degree of LL.D. The following offices were offered to him at various times and declined, viz.: Commandant of Cadets at the Virginia Military Institute in 1860; charge of the Water Department of the City of Baltimore under Mayor Malsted; Superintendent of the United States Military Academy, once by General Sheridan and once by General Schofield; President of the University of West Virginia, in 1895; Superintendent of the Coast and Geodetic Survey, by President Cleveland, and membership in the Isthmian Canal Commission, by President McKinley.

William Price Craighill was elected a Member of the American Society of Civil Engineers on October 7th, 1885; he served as a member of the Board of Direction in 1892 and 1893; and was elected President in 1894, during his term of office as a Director; he was elected an Honorary Member on March 23d, 1896.

It is probably within the truth to say that no epoch in General Craighill's career gave him more pleasure and satisfaction than his official connection with the American Society of Civil Engineers, first as a Director and finally as its President; and it certainly is a fact that this connection with the Society resulted largely to its good, and brought about many permanent and delightful friendships.

In connection with the work of the Society, he was particularly attentive to apparently trivial details, that is, when such details appeared to have an ultimate bearing of importance; on the other hand, when no subsequent advantage was apparent, he was perhaps unequaled in his ability to "side-track" such matters, without, however, appearing brusque or offensive in manner. As is frequently the case where routine matters must be passed upon by a board composed of many men, the tendency is to fritter away time to no useful purpose. Early in the General's service as a Director it was made evident that he had little in common with such discussions, and could confidently be counted on to "vote first and talk afterward." Therefore, as might be expected, he was a business executive of the first order, and it was a delight to sit with him in the Board meetings of the Society.

As a presiding officer, he was exceedingly efficient, combining in a rare degree, seldom equaled, affability, tolerance, tact, dignity, retentiveness of memory, and a swift comprehension of a controverted point. A decision never appeared to be made too quickly; nor was there any uncertainty when rendered.

It is believed that no officer of the Society made, in so few years, so many friends among the younger members of the Society; for, to the young engineer, his courteous and kindly attention was equally a delight to the recipient and to the observer.

He was in all respects well-rounded—a man of many parts. Although bound by the traditions of many years of military service, he was yet peculiarly open-minded in recognizing the necessary differences which have existed, and must exist, in the performance of engineering work under private or individual control and Governmental control. He undoubtedly possessed the attributes essential to success in any walk of life, wherein he would as surely have made his mark as he did in that branch of the Engineering Profession which he adorned so conspicuously.

General Craighill was a member of various committees of the Society, and it may serve to illustrate both the simplicity of his character and his keen discernment of a useful purpose to mention his connection with the Committee which designed the present Society badge. Here, on the face of the matter, the duty was of merely nominal importance, yet he devoted much thought to the subject, and made many suggestions. It is well remembered how, at one of these meetings, he referred with evident gratification to his West Point class pin, the class of 1853, which bore the device of a mortar, with a triangular pile of balls stacked in front of it, and over all the words, "We separate for service." The point he was making was that he believed this sentiment, in his own case at least, had proved to be a potentiality for good, and consequently the same might result from the work in hand. After the present plain lettered badge had been adopted by the Society, he took the keenest interest in having the decision of the Committee carried out, that it should be produced as nearly mechanically perfect as would be possible by available processes and manual skill.

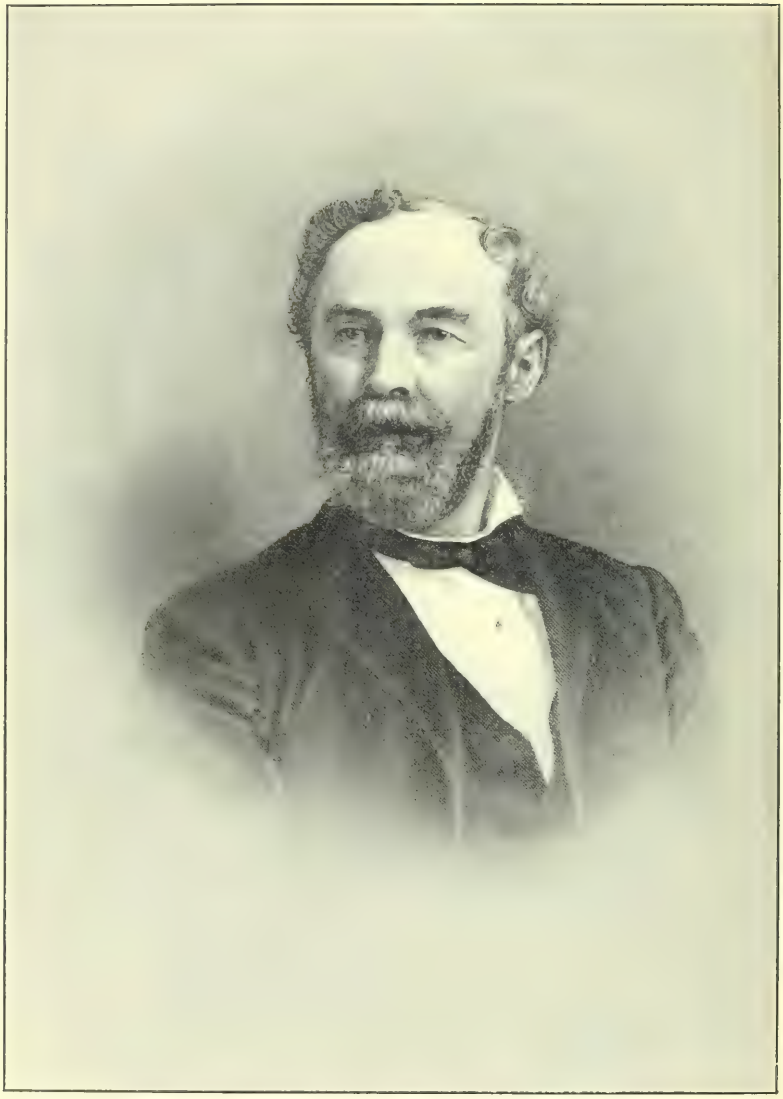
Thus it can verily be believed that in all matters, great or small, his heart was bound in the Society to the end that its influence and prestige might be broadened, the position of the engineering profession be enhanced, and that we, who "separate for service," might with pride and honor carry its insignia of membership.

This brief and all too inadequate tribute to the General's association with the Society may be best closed by his own estimate of the honors it had conferred upon him, for, by his expressed instructions, the following inscription is to be cut upon his tombstone:

WILLIAM PRICE CRAIGHILL,
Brigadier-General and Chief of Engineers,
United States Army;
Honorary Member and Past-President,
American Society of Civil Engineers.

His was a true and loving heart, strong with human feeling. A fitting crown to such a life and character is enduring memory among one's friends and associates. It is certain that he himself would have preferred no honor to his memory more acceptable than this, which, and especially from the members of the Society who had the pleasure and honor of his acquaintance, is abundantly assured.

"Such graves as his are pilgrim's shrines,
Shrines to no code or creed confined."



Chas H. Mann

CHARLES HERMANY, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1908.

Charles Hermany was born in Lynn Township, Lehigh County, Pennsylvania, on October 9th, 1830. He was the son of Samuel and Salome Wannemacker Hermany. His ancestors were of Swiss, French, and English stock, who had emigrated to the United States as early as 1700, and settled in the southeastern part of Pennsylvania. They were agriculturists, and, with their descendants and contemporaries, developed the district from a wilderness to ideal farming land. The farm on which Mr. Hermany was born still belongs to the family, having been deeded to his ancestor, George Philip Hermany, by William Penn.

Mr. Hermany received his early education in the public schools of his home town, where he developed a decided taste for mathematics. Having determined to adopt the Profession of Civil Engineering as his life work, he entered Minerva Seminary, at Easton, Pennsylvania, and was graduated therefrom in two years. Financial conditions prevented him from continuing his collegiate education, and for three years he taught school, at the same time continuing his studies in mathematics and in his chosen profession with a determination which characterized all his work in later life.

In May, 1853, he went West and found employment in the City Engineer's Department of Cleveland, Ohio. Here he was engaged for four years, gradually advancing to the position of First Assistant to the City Engineer, the late T. R. Scowden, M. Am. Soc. C. E. When, in 1857, Mr. Scowden, as Chief Engineer of the Louisville Water Company, took charge of the construction of the water-works system in Louisville, Kentucky, Mr. Hermany went with him as his Principal Assistant. In this capacity he served during the entire construction period and until the works were completed and put in operation in October, 1860. On January 1st, 1861, Mr. Hermany was appointed Chief Engineer and Superintendent of the water-works, which position he held continuously up to the time of his death.

The growth of the city soon proved that the works, as first constructed, were entirely inadequate, and this necessitated their general enlargement. To Mr. Hermany was assigned the duty of designing and constructing a new pumping station on the river bank, which would secure an abundant supply from the river, the stages of which varied more than 40 ft. between high and low water. The treacherous condition of the ground necessitated the sinking of a pneumatic caisson, in

* Memoir prepared by the following Committee: G. H. Benzenberg, Past-President, Am. Soc. C. E., R. Montfort, and Webster Gazlay, Members, Am. Soc. C. E.

order to secure a firm foundation for the pump-house and the pumping engine, as well as to provide for an inlet chamber. The contract was let to an experienced and responsible firm, which, however, finding the work of a hazardous nature, refused to proceed with it. This condition obliged Mr. Hermany to prosecute the work by day labor, requiring his personal supervision and utmost skill for a number of years. In this work he was successful, as he was also in the installation of pumping machinery designed principally by himself, which, at the time of its test run, attracted general attention because its high economic efficiency exceeded all known records. In addition to a betterment of the distribution system, he designed and constructed the Crescent Hill Reservoirs. His last great work was to design and construct the filter plant, including the covered clear-water basin and the high-service pumping station, with its equipment of pumping machinery.

Being extremely careful and prudent, Mr. Hermany conducted a series of exhaustive experiments, lasting more than a year, in order to determine the best method of treating the river water in the process of proposed purification. Very particular in all matters, he practically worked out every detail of all the work which he designed and constructed, the specifications which he prepared for his large pumping engine being an evidence of the minutiæ to which he carried his work. As exacting as he was in these matters, so was he in his requirement of the fulfilment of every obligation on the part of contractors. Faithful to every duty, he insisted upon the same compliance from everyone else.

As an engineer Mr. Hermany was resourceful in every emergency, which enabled him to meet successfully any contingency, as he often had occasion to demonstrate, perhaps at no time so signally as when he promptly restored the works to service by converting the remnants of a tall stand-pipe into a great air chamber, after it had been demolished by a disastrous tornado which swept over Louisville in 1890.

These qualities, as well as his skill, attained for him an eminence in his profession which resulted in his counsel and advice being sought by the water departments of many cities throughout the country. He designed the water-works at Frankfort, and at Bowling Green, Kentucky, and the pumping station at Evansville, Indiana. He also served, in different capacities, the water-works of Nashville, Memphis, St. Louis, Kansas City, Milwaukee, Providence, Chicago, and, finally, as Consulting Engineer during the construction of the new water-works of Cincinnati, Ohio.

Mr. Hermany possessed administrative abilities to a very high degree, which he manifested in conducting the office of Superintendent of Water-Works for more than forty-seven years to the perfect satisfaction of the Water Board and of every water taker, the confidence and regard of all of whom he always possessed.

His reputation for honesty and integrity was ever proverbial, and though, at times, he was impatient and blunt when he suspected unfairness or deceit—for he hated all sham and sordidness—he was at heart scrupulously just, as well as kind, generous, and thoughtful. Courteous and affable to everyone, he was deeply true and loyal to those who were privileged to be enumerated among his friends. The great affection and high esteem in which he was held by the members of his profession, his fellow-citizens, and his neighbors was shown by the great number who were present at the last sad rites, by the immense number of floral tributes under which the casket was buried, and in the last words which were spoken over his remains:

"To discover one's talents, to find one's place in the economy of the world, to put one's powers to the highest use in the service of one's fellow-men, is to merit the approbation of the Judge of all. This is what Mr. Charles Hermany did. He served this community with a faithfulness and devotion to duty that it would be difficult to equal and impossible to excel."

On December 19th, 1864, Mr. Hermany married Miss Sallie Adams, only daughter of Daniel and Sophia (Perdue) Adams. Forty-three years of happy life followed this union, during which seven children blessed their home, three of whom, Irene H., the wife of Mr. Owen Gathright, of Louisville, Emily H., the wife of Mr. James P. Donahue, of Davenport, Iowa, and Madeline, an unmarried daughter, together with his wife, survive him. During the last nine years Mr. Hermany lost his three sons and his youngest daughter, Hetty May, wife of Mr. James L. Smyser, of Louisville. These losses were severe blows to him, from the effects of which he never fully recovered.

Mr. Hermany's health was generally good, and he was ill for only a few days before his death. It had been his intention to attend the Annual Meeting of the Society in January, 1908, but, a few days before, he contracted a bad cold which rapidly developed into pneumonia, involved both his lungs, and resulted in his death within a very few hours, he having been confined to his home less than three days and to his bed less than one day.

A sad feature in connection with his death was the fact that, after many years of earnest and perplexing labor, he was about to complete the filtration works, and this was to round out his professional labors after half a century of faithful, zealous, and devoted services to his home city.

Mr. Hermany was a remarkably self-reliant man, and always exhibited a consciousness of faith in his judgment when once he had arrived at a conclusion. He was a constant reader, and was endowed with splendid intellectual gifts, possessing a mind which was quick to discern and independent in its deductions.

Mr. Hermany never held any political office, though he was always

deeply interested in questions which affected the welfare of his country, state or city, and he did not hesitate to give expression to his views. He was a loyal member of the Unitarian Church of the Messiah, and, for thirty years, was a member of its Board of Trustees. He was the first President of the Engineers' and Architects' Clubs of Louisville, and was re-elected for five consecutive terms.

On January 6th, 1869, Mr. Hermany was elected a Member of the American Society of Civil Engineers. In November, 1879, he was elected and served as a member of the Board of Direction and, in 1891, as Vice-President. In 1904, he was elected President of the Society, and presided at the meetings of the International Engineering Congress, held in St. Louis during the Louisiana Purchase Exposition. During this Congress he was elected an Honorary Member of the Institution of Civil Engineers of Great Britain.

He was also a Member of the Franklin Institute, and of the American Water-Works Association, as well as of the Salmagundi and Filson Clubs, of Louisville, the leading literary organizations of that city.

WALTER LORTON DUSENBERRY, Jun. Am. Soc. C. E.*

DIED MAY 13TH, 1909.

Walter Lorton Dusenberry, the son of Edmund and Caroline Elizabeth (Bennett) Dusenberry, was born in New York City, on February 9th, 1862. On his father's side Mr. Dusenberry was descended from the Von Doesenbury family, who settled in New Amsterdam early in the seventeenth century. Members of the family served with distinction in the Revolutionary War. His mother's family was also of Dutch and Huguenot descent, having settled on Long Island two centuries ago.

Mr. Dusenberry received his early education at Anthon's Grammar School, in New York City. Afterward, he entered the School of Mines of Columbia University, from which he was graduated in 1884 with the degree of E. M.

For some time after his graduation, Mr. Dusenberry was connected with the United States Coast and Geodetic Survey, and was also engaged in mining engineering in the western part of the United States and in Mexico. In 1887 he received an appointment on the New Croton Aqueduct, in New York City, being promoted later for valuable services rendered. From 1889 to 1895 he served as Assistant Engineer in the Department of Public Parks. From 1895 to June, 1897, Mr. Dusenberry was employed, except for a brief interval, in the Department of Public Parks and the Department of Highways, of New York City, in charge of the opening and paving of streets. He was then appointed Engineer to the Commissioners of Accounts, and on leaving that office in 1903, he engaged in expert mining engineering work, and in the examination of properties in the Southwest and in Mexico.

Mr. Dusenberry was married on May 11th, 1899, to Nina Jay Wright, who survives him. He was a member of the Huguenot Society and the Sons of the Revolution, and was also a Mason. His personal character was of the highest, and he was devoted to his profession.

Mr. Dusenberry was elected a Junior of the American Society of Civil Engineers on November 5th, 1890.

* Memoir prepared by the Secretary from material furnished by Mr. Dusenberry's family.

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William P. Morse
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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXV—No. 10

PRESENTED TO
N CIVIL
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BY
WILLIAM P. MORSE

December 1909

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OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 10

DECEMBER, 1909

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NEW YORK 1909

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American Society of Civil Engineers

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ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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CABLE ADDRESS....."Ceas. New York."

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

November 17th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present also, 140 members and 24 guests.

A paper by Julio F. Sorzano, M. Am. Soc. C. E., entitled "Water Supply for the Lock Canal at Panama" was presented by the Secretary. The paper was discussed orally by C. E. Grunsky, M. Am. Soc. C. E., and communications on the subject from Messrs. H. F. Hodges and Theodore Paschke, were presented by the Secretary.

A paper by F. E. Trask, M. Am. Soc. C. E., entitled "The Outlet Control of Little Bear Valley Reservoir," was presented by title.

The Secretary announced the death of Albert Stanley Riffle, elected Member May 7th, 1890; died October 28th, 1909.

Adjourned.

December 1st, 1909.—The meeting was called to order at 8.30 p. m.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 212 members and 42 guests.

A paper by James H. Brace and Francis Mason, Members, Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The Cross-Town Tunnels," was presented by Mr. Mason and illustrated with lantern slides.

The paper was discussed by Edward Wegmann, M. Am. Soc. C. E.

The Secretary announced the election of the following candidates by the Board of Direction on November 30th, 1909:

As Honorary Member.

Hiram Francis Mills, Lowell, Mass.

As Members.

Lynn Leroy Davis, Buffalo, N. Y.
Archibald Stewart Downey, Seattle, Wash.
Harry Madera Gould, Nashville, Tenn.
Louis Jenison Hotchkiss, Chicago, Ill.
Tien Yow Jeme, Peking, China.
Hudson Flack Layton, Pittsburg, Pa.
William Augustus McCalla, Decatur, Ala.
Charles Trinder Middlebrook, Albany, N. Y.
Abraham Moreau Reynolds, Newark, N. J.
Charles Alfred Sheppard, Edwardsville, Ill.
Francis Pitt Smith, New York City.
William Dowlin Tyler, Dante, Va.
Edward Van Winkle, New York City.
Francis Potts Witmer, East Orange, N. J.

As Associate Members.

Lourenco Baeta-Neves, Bello Horizonte, Brazil.
Geert Blaauw, New York City.
George Glover Blackmore, New York City.
Irving Clinton Brower, Nevada, Mo.
James Watson Davenport, Memphis, Tenn.
Benjamin Herman Davis, Hoboken, N. J.
James Moore Farrin, Camaguey Cuba.
Andrew Jackson Fisk, Jr., St. Ignatius, Mont.
Russell John Hawn, Fordwick, Va.

John Edward Jennings, Milliken, N. Y.
Thomas Edward Jewett, Houston, Tex.
Walter Alpheus Jones, Titusville, Pa.
Reuben Sylvester Peotter, Mercedes, Tex.
Walter Edward Sanford, Elmhurst, N. Y.
Robert Warren Shelmire, Cleveland, Ohio.
Jay Varnum Wescott, Chicago, Ill.

As Associates.

James Vance Howe, Morgantown, W. Va.
Holger Struckmann, St. Louis, Mo.

As Juniors.

Harold Affleck Brainerd, Elmira, N. Y.
Walter Day Cater, Yorktown, Va.
Harold Bowen Edmundson, Thornton, R. I.
Ralph William Hazen, Manila, Philippine Islands.
Walter William Huff, Cleveland, Ohio.
Alfred Kessinger McCampbell, Billings, Mont.
William John Millard, Guatemala City, Guatemala.
Hugh Miller, Potsdam, N. Y.
Edgar Alva Norwood, Rockport, Mass.
Clifford French Phillips, St. Louis, Mo.
Hunter Imboden Snyder, Nashville, Tenn.
Edward Wording Stearns, Wilkinsburg, Pa.
Charles Sumner Stewart, Morris, Minn.
Jay William Toms, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on November 30th, 1909:

From Associate Member to Member.

Norman Freed Brown, Pittsburg, Pa.
Edwin Clifford Finley, St. Louis, Mo.
Henry Neely Ogden, Ithaca, N. Y.
DeWitt Lee Reaburn, Surrey, Cal.
John Martin Schreiber, Newark, N. J.
Wilford Ashford Thompson, East St. Louis, Ill.

From Junior to Associate Member.

Wilbur Howard Fisher, Twin Falls, Idaho.
Camden Page Fortney, Gatun, Canal Zone, Panama.

The Secretary announced the death of James Gardner Sanderson, elected Member, July 6th, 1881; died November 15th, 1909.

Adjourned.

December 15th, 1909.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1910. Two papers have been set down for discussion: "The New York Tunnel Extension of the Pennsylvania Railroad: The East River Tunnels," by James Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E., and "The New York Tunnel Extension of the Pennsylvania Railroad: Contractors' Plant for East River Tunnels," by Henry Japp, M. Am. Soc. C. E.

OF THE BOARD OF DIRECTION

(Abstract)

November 30th, 1909.—Vice-President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Churchill, Endicott, Hazen, Hodgdon, Knap, Schneider, Swensson, Thompson, Tillson, and Williams.

The subscription price to *Proceedings* was fixed at $8 per annum for the future, to begin January 1st, 1910.

A special subscription rate to students of any technical school, for *Proceedings,* was fixed at $4.50 per annum.

Hiram Francis Mills was elected an Honorary Member.

Ballots for membership were canvassed, resulting in the election of 1 Honorary Member, 14 Members, 16 Associate Members, 2 Associates, and 14 Juniors, and the transfer of 2 Juniors to the grade of Associate Member.

Six Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

January 5th, 1910.—8.30 P. M.—At this meeting a paper by Horace J. Howe, M. Am. Soc. C. E., entitled "Notes on the Replacing of the Superstructure of the Harlem Ship Canal Bridge," will be presented for discussion.

This paper was printed in *Proceedings* for November, 1909.

Wednesday and Thursday, January 19th and 20th, 1910.—The Fifty-seventh Annual Meeting will be held. The Business Meeting will be called to order at 10 o'clock on Wednesday morning at the Society House. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

Arrangements for the Annual Meeting have been placed in the hands of a committee composed of Messrs. T. Kennard Thomson, Thaddeus Merriman, and Charles Warren Hunt.

February 2d, 1910.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Underpinning the Cambridge Building, New York City," by T. Kennard Thomson, M. Am. Soc. C. E., and "Building Agreements," by William B. Bamford, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

February 16th, 1910.—8.30 P. M.—A paper entitled "The Effect of Alkali on Concrete," by George Gray Anderson, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

SUBSCRIPTION PRICE TO THE PUBLICATIONS OF THE SOCIETY

The following subscription rates have been fixed by the Board of Direction for the publications of the Society:

Proceedings, ten Numbers per annum, $8.

Transactions, four Volumes per annum, $12. Price for single volumes, $4.

On the above prices there is a discount of 25% to members who desire extra copies of any of these publications, to Libraries, and to Book-dealers.

A special subscription rate has been fixed by the Board for the *Proceedings* of the Society for the benefit of Students in Technical

Schools. This rate is $4.50 per annum, and is available to any *bona fide* student of any technical school.

MEETINGS OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS, AM. SOC. C. E.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet and weekly informal luncheons. The former are held at 6 P. M., at the Fairmont Hotel, on the third Friday of February, April, June, August, October, and November, and also on the third Wednesday of December, the latter being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., Assoc. M. Am. Soc. C. E., 623 Wells Fargo Building.

The by-laws of the Association provide for the extension of hospitality to any members of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

PAPERS AND DISCUSSIONS

The first three Volumes of *Transactions* for 1909 (Volumes LXII, LXIII, and LXIV) have been issued. The fourth volume will be issued on December 31st.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has to be established.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availablity for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the

opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Colorado Association

(Abstract of Minutes of Meeting)

November 13th, 1909.—The regular meeting of the Colorado Association of Members, Am. Soc. C. E., was called to order at 8 P. M.; George G. Anderson, Vice-President, in the chair; H. J. Burt, Secretary, and present, also, 8 members and 7 guests.

The reading of the Minutes and other regular business was dispensed with, on account of the special character of the programme.

An address on "Work" was made by Mr. A. D. Parker, Vice-President of the Colorado and Southern Railroad Company, and the subject was discussed by a number of those present.

A resolution of thanks was unanimously voted to Mr. Parker.

Adjourned.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From November 9th to December 6th, 1909)

DONATIONS*

CYRUS HALL McCORMICK, HIS LIFE AND WORK.

By Herbert N. Casson. Cloth, 8¼ x 5½ in., illus., 12 + 264 pp. Chicago, A. C. McClurg & Co., 1909. $1.50 net.

In the author's opinion anyone who wishes to understand the making of the United States must read the life of Cyrus Hall McCormick, which spanned the heroic period of the industrial advancement of the United States and represented the replacing of muscle with machinery, and the establishment of better ways and better times in farm and city alike. The Contents are: The World's Need of a Reaper; The McCormick Home; The Invention of the Reaper; Sixteen Years of Pioneering; The Building of the Reaper Business; The Struggle to Protect Patents; The Evolution of the Reaper; The Conquest of Europe; McCormick as a Manufacturer; Cyrus H. McCormick as a Man; The Reaper and the Nation; The Reaper and the World; Give Us This Day Our Daily Bread; Index.

CONCRETE INSPECTION.

A Manual of Information and Instructions for Inspectors of Concrete Work, with Standard and Typical Specifications. By Charles S. Hill. Cloth, 6 x 3½ in., illus., 8 + 179 pp. Chicago and New York, The Myron C. Clark Publishing Co.; London, B. & F. N. Spon, Ltd., 1909. $1.00 net.

This volume consists, it is stated, of a series of rules and directions to be followed in inspecting concrete work, with brief explanations of the reasons for each rule and of their importance. The author believes that the subject-matter is based on sound practice and acknowledged authority. The Contents are: Inspection of Concrete Materials; Inspection of Proportioning and Mixing; Inspection of Form Work; Inspection of Reinforcement; Inspection of Concreting; Inspection of Sidewalk Construction; Inspection of Molding and Driving Concrete Piles; Inspection of Cast Concrete Work; Standard and Typical Specifications; Index.

A TEXT BOOK ON GRAPHIC STATICS.

By Charles W. Malcolm, Assoc. M. Am. Soc. C. E. Cloth, 9 x 6 in., 12 + 316 pp. New York and Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1909. $3.00 net.

It has been the aim of the author to give proofs of the principles stated, with full explanations of the construction. No attempt has been made, however, to give elaborate solutions which have little or no practical application. Most of the material in Parts I and II, the author states, has been used in printed form by his students for several years, and he expresses the hope that Parts II, III, and IV will be found of assistance to the practicing engineer. The Contents are: Part I, General Principles—Definitions; Concurrent Forces; Non-Concurrent Forces; Moments; Center of Gravity of Areas; Moment of Inertia. Part II, Framed Structures, Roof Trusses—Definitions; Loads; Reactions; Stresses in Roof Trusses; Wind Load Stresses; Stresses in Cantilever and Unsymmetrical Trusses—Maximum Stresses; Counterbracing; Three-Hinged Arch; Stresses in a Transverse Bent of a Building; Miscellaneous Principles. Part III, Beams—Bending Moments, Shears, and Deflections in Beams for Fixed Loads; Maximum Bending Moments and Shears in Beams for Moving Loads. Part IV, Bridges—Types of Bridge Trusses; Loads; Stresses in Trusses Due to Uniform Loads; Influence Diagrams, and Positions of Engine and Train Loads for Maximum Moments, Shears, and Stresses; Maximum Moments, Shears, and Stresses Due to Engine and Train Loads; Index.

SOLID BITUMENS:

Their Physical and Chemical Properties and Chemical Analysis, Together With a Treatise on the Chemical Technology of Bituminous Pavements. By S. F. Peckham. Cloth, 9 x 6 in., illus., 8 + 324 pp.

* Unless otherwise specified, books in this list have been donated by the publisher.

New York and Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1909. $5.00 net.

This book, states the author, represents personal experiences and experiments prosecuted almost continuously for forty years by nearly every method of investigation proposed in the United States and Europe, and the reading of nearly everything published relating to bitumens, from Boussingault's Memoir to the present time. The author's aim has been to offer everything of value relating to the subject, and reference has been made to original memoirs as far as possible. The Contents are: Part I, The Natural History of Bitumens—Introduction; Geographical Distribution of Bitumens; Origin of Bitumens; The Classification of Bituminous Substances; The Derivation of Natural Solid Bitumens; The Derivation of Bituminous Rocks; The Derivation of Artificial Solid Bitumens. Part II, The Chemistry of Bitumens—General Considerations Concerning the Chemistry of Solid Bitumens; Boussingault's Memoir Upon the Composition of Bitumens; The Use of the Words, Petrolene, Asphaltene, Etc.; The Ultmate Analysis of Solid Bitumens; The Proximate Analysis of Solid Bitumens; The Technical Analysis of Solid Bitumens; Special Chemical and Physical Methods of Analysis by Which Solid Bitumens may be Recognized and Distinguished; Miscellaneous Methods Applied to Street Mixtures and Surfaces, Bituminous Paving Blocks, Bituminous Concrete, Wood Paving Blocks, Cements, Cement Mortars and Concrete, Etc. Part III, The Physical Properties of Solid Bitumens—Specific Gravity, Etc. Part IV, Chemical Technology of Bituminous Streets—Historical Introduction; A Modern Street; Bitulithic Streets; The Oiled Roads and Streets of the Pacific Coast; Asphalt Blocks; Wood Blocks; Conclusions; Index of Subjects; Index of Names.

LAVORAZIONA E TEMPERA DEGLI ACCIAI.

Indurimento Superficiale del Ferro e Cementazione. By Arturo Massenz. Seconda Edizione. Cloth, 6 x 4 in., illus., 16 + 118 pp. Milano, Ulrico Hoepli, 1909. 2 lira.

It has been the author's purpose to produce a popular book, written by a practical person, which will be an aid, especially to the worker of metals, in the method of forging and tempering steel. This second edition has been enlarged by filling in the omissions in the first, and by the addition of a special study on tempering furnaces, etc. A partial list of contents follows: Che cosa è l'acciaio?; Esigenze alle quali dovranno rispondere gli utensili da construire; Qualità d'acciaio preferirsi; Cambiamenti ai quali andrà soggetto l'acciaio nelle varie lavorazioni; Regole onde evitare gl'insuccessi nella tempera; Ricottura dopo la tempera; Cause principali determinanti gl'insuccessi nella lavorazione e nella tempera; Forno speciale per tempera, con bagno riscaldato elettricamente; Trattamento degli acciai secondo lo scopo pel quale sono destinati; Saldatura dell' acciaio, ecc.; Indurimento superficiale del ferro; Cementazione.

ELECTRIC POWER CONDUCTORS.

By William A. Del Mar. Cloth, 7¼ x 5¼ in., illus., 6 + 339 pp. New York, D. Van Nostrand Company, 1909. $2.00 net.

This book is intended to present a clear account of all engineering conditions which affect the purchase and use of power conductors. The author states that it will be found practical and up to date, being based on notes prepared for his own use, and containing nothing from any published data which has not been thoroughly studied and found to be reliable. The text is made as brief as possible, it is declared, and where explanation or theoretical discussion is advisable, it is supplemented by appendices. The sections on Alternating Current Feeder Calculations and Stress in Spans are written by Dr. Harold Pender. The first nine chapters pertain to wires and cables. The Contents are: Materials and Gauges; Electrical Properties; Insulation and Insulated Conductors; Determination of Size for Given Voltage Drop and Power Loss; Determination of Size for Given Stress in Spans; Specifications; Testing Wire and Cable; Installation; Depreciation and Deterioration; Third Rail Circuits; Rail Bonds; Tables of Inductance, Reactance and Capacity. Appendices: I, Basis of B. & S. Gauge; II, Basis of Skin Effect of Carrying Capacity Formulæ; III, Method of Calculating Thickness of Rubber Insulation; IV, Basis of Direct and Alternating Current Transmission; V, Basis of Formulæ for Stresses in Spans; VI, Explanation of Specifications; VII, Basis of Tables of Inductance; Index.

THE COLLECTION AND DISPOSAL OF MUNICIPAL WASTE.

By William F. Morse. Cloth, 9¼ x 6 in., illus., 17 + 462 pp. New York, The Municipal Journal and Engineer, 1909. $5.00.

In this work the author presents, it is stated, in as compact a form as possible, data gathered by him during nearly twenty-one years of continuous work, together

with information collected from scattered reports, papers, and a great variety of other sources. It is the author's purpose to give an historical sketch of disposal works in North American communities from the time the subject first assumed general importance (about 1885) to date. It is his further purpose to present an account of the various methods of waste collection and disposal in use in the United States and a comparison of the older with the more modern systems. There is also a short account of the progress of refuse disposal in other countries by W. Francis Goodrich, with short articles on "The Disposal of City Waste by Gas Producers," by F. C. Tryon, and "The Utilization of City Ashes," by C. H. Koyl The book is divided into five parts, as follows: Part I, The Municipal Waste of American Towns; Part II, The Disposal of American Municipal Waste by Crematories and Incinerators; Part III, The Disposal of Waste by British Destructor Systems; Part IV, The Disposal of Waste by Reduction and Extraction Process; Part V, The Utilization of Municipal Waste.

RADIATION, LIGHT AND ILLUMINATION.

A Series of Engineering Lectures Delivered at Union College. By Charles Proteus Steinmetz. Compiled and Edited by Joseph Le Roy Hayden. Cloth, 9½ x 6 in., illus., 12 + 305 pp. New York, McGraw-Hill Company, 1909. $3.00.

The author states that these lectures are intended, not merely as a textbook on illuminating engineering or on the physics of light and heat radiation, but as an exposition, from the engineering point of view, of that knowledge of light and radiation which every educated man should possess. For this purpose they have been given in such form as to need no special knowledge of mathematics or engineering, and they describe the phenomena in plain language, with the exception of Lectures X and XI, which, by their nature, are somewhat mathematical, and are intended more particularly for the illuminating engineer. It is also stated that the lectures have been revised to date, and the important results of the National Bureau of Standards, contained in its recent bulletins, have been fully utilized. The compiler is authority for the statement that, in describing the experiments, numerical and dimensional data have been given, and the illustrations have been drawn to scale as far as possible, so as to make the repetition of the experiments convenient. The Contents are: Nature and Different Forms of Radiation; Relation of Bodies to Radiation; Physiological Effects of Radiation; Chemical and Physical Effects of Radiation; Temperature Radiation; Luminescence; Flames as Illuminants; Arc Lamps and Arc Lighting; Measurement of Light and Radiation; Light Flux and Distribution; Light Intensity and Illumination; Illumination and Illuminating Engineering; Physiological Problems of Illuminating Engineering; Index.

THE TECHNICAL PRESS INDEX, JANUARY, 1908, TO JUNE, 1909, INCLUSIVE.

Compiled in Cumulative Form from the Technical Press Index Published Monthly in *The Engineering Digest.* Cloth, 10 x 7¼ in., 24 + 373 pp. New York, The Technical Literature Company; London, Archibald Constable & Company, Limited, 1909. $3.00 net. (Donated by the Engineering News Publishing Co.)

The preface states that the items taken from the monthly index of *The Engineering Digest* have been rearranged, reclassified, and cross-indexed, for this first cumulative volume of the Technical Press Index, and that it is believed to contain reference to every important paper and article published, during the time covered, in the regular engineering periodicals of the United States, Canada, and Europe, and in the proceedings of the engineering and technical societies. All articles of evident temporary, local, or individual interest have been omitted, it is said, and an effort has been made to cover the field of engineering science in a permanently valuable manner. Where ambiguous titles have been used by authors, or where amplification seemed desirable, descriptive notes have been added, and in every case the number of words, number of figures or illustrations, and price are given. Beside the cross-indexing there has been introduced also a key, or index to the Index, consisting of an alphabetical list of technical catch-words or key-words, giving the different pages on which a subject will be found, many of the topics being brought out under several different heads. The main divisions, or heads, are as follows: Architecture; Automobiles and Aerial Navigation; Civil Engineering; Economics and Education; Electrical Engineering; Industrial Technology; Marine Engineering; Mechanical Engineering; Metallurgy; Mining Engineering; Municipal Engineering; Railway Engineering.

Gifts have also been received from the following:

Am. Ry. Master Mechanics Assoc. 1 bound vol.
Arnold, B. J. 1 bound vol.
Assoc. of Licensed Automobile Mfrs. 1 pam.
Bartholdt, Richard. 1 pam.
Boston, Mass.-Transit Comm. 1 bound vol.
Camden, N. J.-Chf. Engr. of the Water Dept. 1 pam.
Canada-Board of Ry. Commrs. 1 pam.
Canada-Dept. of Marine and Fisheries. 1 vol., 2 pam.
Canada-Geol. Survey Branch. 4 pam.
Cincinnati Univ. 1 pam.
Committee of One Hundred on National Health. 1 pam.
Conway, George R. G. 1 pam.
Dist. of Columbia-Engr. Comm. 2 vol.
Eng. Assoc. of the South. 1 pam.
Fairmount Park Art Assoc. 8 pam.
Fanning, John T. 1 vol.
Fendall, Benjamin T. 8 bound vol.
Fidelity & Casualty Co. of New York. 2 pam.
Hering & Fuller. 1 pam.
Illinois-Bureau of Labor Statistics. 1 bound vol.
Illinois-R. R. and Warehouse Comm. 1 bound vol.
Illinois-State Geol. Survey. 1 bound vol.
Illinois-Water Supply Assoc. 1 bound vol.
Incorporated Assoc. of Municipal and County Engrs. 1 bound vol.
Institution of Civil Engrs. 1 bound vol.
Iowa-Board of R. R. Commrs. 1 bound vol.
Joint Committee on Conservation. 1 pam.
Langthorn, Jacob E. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
McMinn, Thomas J. 1 bound vol., 1 vol.
Manchester Assoc. of Engrs. 2 bound vol.
Master Car Builders' Assoc. 1 bound vol.
Maury, Dabney H. 2 pam.
Mich.-R. R. Comm. 1 bound vol.
Mobile & Ohio R. R. Co. 1 pam.
Montreal, Canada-City Clerk. 1 bound vol.
Morris, Marshall. 1 bound vol.
National Elec. Light Assoc. 3 bound vol.
New York City-Board of *City Record*. 4 bound vol.
New York City-Board of Health. 1 bound vol.
New York City-Dept. of Docks and Ferries. 1 bound vol.
New York State-Commrs. of the State Reservation at Niagara. 1 pam.
New York State-Public Service Comm., First Dist. 4 pam.
Norboe, P. M. 1 pam.
North Dakota-State Geol. Survey. 2 vol.
North-East Coast Institution of Engrs. and Shipbuilders. 1 bound vol.
Ohio Elec. Light Assoc. 1 bound vol.
Philadelphia, Pa.-Bureau of Surveys. 1 bound vol.
Philadelphia, Pa.-Bureau of Water. 2 vol.
Pratt Inst. Free Library. 1 pam.
Roberts & Abbott Co. 2 pam.
Rogers, J. W. 1 pam.
Sanderson & Porter. 1 pam.
South African Assoc. of Engrs. 1 vol.
South Australia-Rys. Commr. 1 pam.
Southern Pacific Co. 2 pam.
Svenska Teknologföreningen. 1 pam.
Syracuse, N. Y.-Intercepting Sewer Board. 1 pam.
Union Univ. 1 pam.
U. S.-Chf. of Engrs. 3 bound vol., 6 pam.
U. S.-Coast and Geodetic Survey. 2 pam.
U. S.-Geol. Survey. 9 pam.
U. S.-Interstate Commerce Comm. 1 pam.
U. S.-Isthmian Canal Comm. 1 vol.
U. S.-Lake Survey Office. 1 pam., 1 map.
U. S.-Office of Exper. Stations. 6 pam.
U. S.-War Dept. 1 bound vol.
Victoria, Australia-Chf. Engr. for Ry. Constr. 1 pam.
Victoria, Australia-State Rivers and Water Supply Comm. 1 pam.
Western Australia-Commr. of Rys. 1 pam.
Williams, Winfield S. 2 pam.

BY PURCHASE

Minutes of Seventh Annual Meeting of the National Conference on Standard Electrical Rules, held at the Engineering Building, 29 West 39th Street, New York City, March 26th, 1909.

The Water Works Directory and Statistics, 1909. Including Index of Officials and List of Associations of Water Engineers. Hazell, Watson & Viney, Ltd., London, 1909.

American Inland Waterways. Their Relation to Railway Transportation and to the National Welfare; Their Creation, Restoration and Maintenance. By Herbert Quick. G. P. Putnam's Sons, New York and London, 1909.

Allgemeine Theorie über die veränderliche Bewegung des Wassers in Leitungen. I. Teil: Rohrleitungen, von Lorenzo Alliévi (Deutsche, erläuterte Ausgabe, bearbeitet von Robert Dubs und V. Bataillard). II. Teil: Stollen und Wasserschloss, von Robert Dubs. Julius Springer, Berlin, 1909.

Die Berechnung von Steifrahmen nebst Anderen Statisch Unbestimmten Systemen. Von Ejnar Björnstad. Julius Springer, Berlin. 1909.

Transactions of the Institution of Mining and Metallurgy. General and Personal Index, Vols. I to XV, 1892–1906. E. & F. N. Spon, Limited, and the Institution of Mining and Metallurgy, London; Engineering and Mining Journal, New York, 1909.

SUMMARY OF ACCESSIONS

(From November 9th to December 6th, 1909)

Donations (including 17 duplicates)	143
By purchase	6
Total	149

MEMBERSHIP

ADDITIONS

(November 10th to December 7th, 1909)

MEMBERS		Date of Membership
ARTHUR, HOWARD ELMER. Chf. Engr., Mosher Mfg. Co., Dallas, Tex.		Nov. 8, 1909
DENCER, FREDERICK WILLIAM. Engr., Lassig Plant, Am. Bridge Co., Wrightwood and Clybourn Aves., Chicago, Ill.	Assoc. M. M.	Feb. 7, 1906 Nov. 8, 1909
ELLERY, NATHANIEL. State Engr., Dept. of Eng., Sacramento, Cal.		Nov. 8, 1909
GODFREY, EDWARD. Structural Engr., Robert W. Hunt & Co., Monongahela Bank Bldg., Pittsburg, Pa.		Oct. 5, 1909
GREGORY, WILLIAM BENJAMIN. Prof. of Experimental Eng., Tulane Univ, and Irrig. Engr., U. S. Dept. of Agri., New Orleans, La.		Nov. 8, 1909
HAZEN, WILLIAM NELSON. Engr., Expanded Metal Eng. Co., Union Bldg., Newark, N. J.	Assoc. M. M.	Dec. 3, 1902 Nov. 8, 1909
KNOX, SCHUYLER BRUSH. Eastern Mgr. and Contr. Engr., Fort Pitt Bridge Works, 45 Broadway, New York City.		Nov. 8, 1909
LALL, CHIRANJI. Head Master, Govt. School of Eng., Anarkali, Nila Gumbad, Mool Chand New Bldg., Lahore, Punjab, India.		Oct. 5, 1909
LAYTON, HUDSON FLACK. Civ. and Cons. Engr., 413 Fourth Ave., Pittsburg, Pa.		Nov. 30, 1909
LORRAINE, MADISON JOHNSON. 2107 Dwight Way, Berkeley, Cal.		Nov. 8, 1909
MCKAY, GEORGE ALBERT. Civ. Engr., U. S. N., U. S. Naval Training Station, Great Lakes, North Chicago, Ill.	Assoc. M. M.	Feb. 4, 1903 Nov. 8, 1909
MORRIS, CLYDE TUCKER. Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.	Assoc. M. M.	Mar. 6, 1907 Nov. 8, 1909
MURALT, CARL LEONARD DE. Prof. of Elec. Eng., Univ. of Michigan; Cons. Engr. (Muralt & Co.), 114 Liberty St., New York City.		Aug. 31, 1909
NICHOLS, CHARLES HART. 42 East 23d St., New York City.		Nov. 8, 1909
REYNOLDS, ABRAHAM MORLAU. Engr. and Supt., Essex County Park Comm., 60 Clifton Ave., Newark, N. J.		Nov. 30, 1909
SCHERER, CLINTON LYTER. City Engr., Beaumont, Tex.		Nov. 8, 1909
SCHNEIDER, EDWARD JOHN. Contr. Mgr., Am. Bridge Co. of New York, 906 Crocker Bldg., San Francisco, Cal.	Assoc. M.	July 9, 1906 Nov. 8, 1909
SMITH, FRANCIS PITT. Chemical and Cons. Paving Engr. (Dow & Smith), 24 East 21st St., New York City.		Nov. 30, 1909

MEMBERS (*Continued*).		Date of Membership.
Van Winkle, Edward. Cons. Patent Engr., The West Street Bldg., New York City		Nov. 30, 1909
Wadsworth, George Reed. Cons. Engr., 19 Congress St., Boston, Mass.	Assoc. M.	Nov. 7, 1906
	M.	Nov. 8, 1909
Wooldridge, Charles Lawson. Superv. Engr., T. A. Gillespie Co., 125 South Lang Ave., Pittsburg, Pa.		Oct. 5, 1909
Yuille, Nathaniel Alston. U. S. Asst. Engr., Box 709, Mobile, Ala.	Assoc. M.	June 6, 1906
	M.	Nov. 8, 1909

ASSOCIATE MEMBERS

Arce, Julius Andrew. Apartado 90, Arequipa, Peru.		Aug. 31, 1909
Armstrong, Roger Wellington. Asst. Designing Engr., N. Y. Board of Water Supply, 299 Broadway, New York City.		Nov. 8, 1909
Atterbury, Charles De La Plane. City Engr., Centralia, Lewis Co., Wash.		Nov. 8, 1909
Baeta-Neves, Lourenco. Chf. Engr., Technical Dept. of Railways, Public Works and Industries. Bello Horizonte, Minas Geraes, Brazil.		Nov. 30, 1909
Barnard, Wilfred Keefer. Asst. Engr., M. of W., San Pedro, Los Angeles & Salt Lake R. R., 2711 Ellendale Pl., Los Angeles, Cal.		Nov. 8, 1909
Beale, Harry Orlando. Care, U. S. Reclamation Service, Fort Shaw, Mont.		Oct. 5, 1909
Berry, Francis Rigdon. Care, Portsmouth, Berkley & Suffolk Water Co., Portsmouth, Va.	Jun.	Dec. 2, 1902
	Assoc. M.	Nov. 8, 1909
Black, Ralph Peters. Div. Engr., K. & M. Ry., Charleston, W. Va.		Nov. 8, 1909
Blood, Charles Frederick. Ridgefield, N. J.		Oct. 5, 1909
Bowie, Clifford Pinkney. Asst. Engr., Associated Pipe Line Co., 1360 Spring St., Berkeley, Cal.		Aug. 31, 1909
Boyd, Butler Bennett. Asst. Engr., San Diego & Arizona Ry., 3210 F St., San Diego, Cal.		Nov. 8, 1909
Collins, Frank Joseph. 600 E. Fayette St., Syracuse, N. Y.		Nov. 8, 1909
Davies, William Gomer. Willows, Cal.	Jun.	Feb. 2, 1904
	Assoc. M.	Nov. 8, 1909
Davis, Benjamin Herman. Asst. Engr., D., L. & W. R. R., Hoboken, N. J.		Nov. 30, 1909
Duder, John. Asst. City Engr., 1080 Third Ave., Salt Lake City, Utah.		Nov. 8, 1909
Evans, John Edward. Engr. of Alignment, Detroit River Tunnel Co., 84 Trumbull Ave., Detroit, Mich.		Nov. 8, 1909
Goodman, Louis. Pres., The Wormser-Goodman Constr. Co., 2 Rector St., Room 520, New York City.		Oct. 5, 1909
Hasbrouck, Oscar. 136 Saratoga Ave., Northside, Cohoes, N. Y.	Jun.	Jan. 3, 1907
	Assoc. M.	Nov. 8, 1909

ASSOCIATE MEMBERS (*Continued*).		Date of Membership.
HAWN, RUSSELL JOHN. Supt., Virginia Portland Cement Co., Fordwick, Va.		Nov. 30, 1909
HAZARD, WILLIAM ABBOTT. Mgr., Lackawanna Bridge Co., Buffalo, N. Y.		Nov. 8, 1909
HIGGINSON, JONATHAN YATES. Res. Engr., N. Y., N. H. & H. R. R., 88 Centre St., New Rochelle, N. Y.	Jun.	Oct. 1, 1904
	Assoc. M.	Nov. 8, 1909
JOHNSTON, CHARLES EUGENE. Div. Engr., Kansas City Southern Ry., Mena, Polk Co., Ark.		June 1, 1909
JONES, FREDERICK ARMISTEAD. Res. Engr., Mo. Pac. Ry. Co., Ft. Gibson, Okla.		Nov. 8, 1909
LONG, CLARENCE BURTON. Augusta, Lewis and Clark County, Mont.		Nov. 8, 1909
MAUPIN, EDGAR STAPLES. Jun. Engr., U. S. Engr. Office, Champagnolle, Ark.	Jun.	Sept. 5, 1905
	Assoc. M.	Nov. 8, 1909
MAYHEW, ALFRED BOARDMAN. Asst. Engr., U. S. Reclamation Service, Boise, Idaho.		Nov. 8, 1909
MINER, JAMES HENRY. Asst. Engr., U. S. Reclamation Service, Montrose, Colo.	Jun.	Mar. 31, 1903
	Assoc. M.	Nov. 8, 1909
MITCHAM, GEORGE NATHAN. Prof. of Civ. Eng., Alabama Poly. Inst., Auburn, Ala.		Nov. 8, 1909
MORROW, FREDERICK EDGAR. 181 La Salle St., 10th Floor, Chicago, Ill.		Nov. 8, 1909
NEWMAN, ARTHUR TRAVER. Care, A. L. Webster, 82 Wall St., New York City.		Nov. 8, 1909
NORCROSS, THEODORE WHITE. Asst. Engr., U. S. Geological Survey, Water Resources Branch, 6 Beacon St., Room 809, Boston, Mass.		Nov. 8, 1909
PLOCK, ALBERT FRANCIS. Chf. Engr., Northwestern Iron Co., Mayville, Wis.		Nov. 8, 1909
PRATT, HENRY BLANCHARD. Asst. Engr., J. R. Worcester & Co.; Res., 750 Lexington St., Waltham, Mass.		June 1, 1909
ROBBINS, HALLET RICE. Civ. and Min. Engr., P. O. Box 51, Seattle, Wash.	Jun.	Nov. 5, 1907
	Assoc. M.	Nov. 8, 1909
RUSCH, HENRI. Chf. Engr., Bldg. Dept., 770 Bayard Ave., St. Louis, Mo.		Nov. 8, 1909
SANFORD, WALTER EDWARD. Asst. Engr., P. T. & T. R. R. Co., Hammond Pl., Elmhurst, N. Y.		Nov. 30, 1909
SMITH, FORREST LEIGH. Township Engr., 102 Smith St., Perth Amboy, N. J.	Jun.	April 30, 1907
	Assoc. M.	Nov. 8, 1909
SPEARMAN, CHARLES. Room 210, Boise City National Bank Bldg., Boise, Idaho		Oct. 5, 1909
SPENCER, LOUIS BERNARD. Civ. and Min. Engr., P. O. Box 499, Ogden, Utah	Jun.	Oct. 31, 1899
	Assoc. M.	Nov. 8, 1909
STRATHMANN, EDWARD CHARLES. Gen. Supt., Bedford Stone & Constr. Co., 814 Traction Bldg., Indianapolis, Ind.		Nov. 8, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
THOMAS, WILLIAM MICHAEL. 1012 Union Trust Bldg., Los Angeles, Cal.	Nov. 8, 1909
UNGER, ERIK BIRGER LUND. Mexican R. R., City of Mexico, Mexico.	Nov. 8, 1909
WALKER, EDWARD MANSFIELD. Engr. of Alignment, Detroit River Tunnel Co., 489 Fort St., West, Detroit, Mich.	Nov. 8, 1909
WHITE, DAVID MILLER. P. O. Box 278, Midland, Tex.	Nov. 8, 1909
WHITING, GEORGE WILLIAM CARLYLE. Asst. Engr., Baltimore Sewerage Comm., 112 E. Preston St., Baltimore, Md. — Jun.	Sept. 5, 1905
— Assoc. M.	Nov. 8, 1909
WILLIAMS, LESTER DENNISON. 702½ Marshall St., Milwaukee, Wis.	Nov. 8, 1909
WOOD, ROBERT WALTER. Asst. Engr. in Chg. of Port Richmond Div., Borough of Richmond, New York City, 13 Van Pelt Ave., Mariner Harbor, N. Y. — Jun.	April 30, 1907
— Assoc. M.	Nov. 8, 1909

ASSOCIATES

CUSHMAN, ALLERTON SEWARD. Asst. Director, Office of Public Roads, Dept. of Agriculture, 1314 Sixteenth St., N. W. Washington, D. C.	Nov. 8, 1909
STRUCKMANN, HOLGER. Third Vice-Pres., Union Sand & Material Co., 6330 Washington Bldg., St. Louis, Mo.	Nov. 30, 1909

JUNIORS

BLUHM, HERMAN WILLIAM. Draftsman, Isthmian Canal Comm., Corozal, Canal Zone, Panama.	Nov. 8, 1909
BOCK, CARL AUGUST. Casilla 43, Oruro, Bolivia.	Aug. 31, 1909
BRAINERD, HAROLD AFFLECK. 360 W. Church St., Elmira, N. Y.	Nov. 30, 1909
CARPENTER, J. C. City Engr. and Surv., Watertown, S. Dak.	Nov. 8, 1909
CHANDLER, EMERSON LAWRENCE. 16 Squire St., New London, Conn.	Nov. 8, 1909
CLIFFORD, WALTER WOODBRIDGE. 60 Oak St., Hyde Park, Mass.	Nov. 8, 1909
DUNAN, GEORGE EDMUND. Asst. Engr., Morey-Faulhaber Constr. Co., Apalachicola, Fla.	Nov. 8, 1909
GRAHAM, GERMAIN PAUL. 280 Quail St., Albany, N. Y.	Nov. 8, 1909
HAYWOOD, CHARLES ELLSWORTH. 37 Van Name Ave., Mariner Harbor, N. Y.	Nov. 8, 1909
HESS, JOHN STRIDER. Box 967, Dawson, Y. T., Canada.	Oct. 5, 1909
LEONARD, OLIVER YEATON. Instrumentman, Porto Rico Irrig. Service, Guayama, Porto Rico.	Nov. 8, 1909
LILLY, RIDGELY CASEY. U. S. Inspector, 3d Dist., Mississippi River Comm., Box 404, Vicksburg, Miss.	Oct. 5, 1909
MISCHKE, ALEXANDER. Jouravlinaya 16, Warsaw, Russia.	Oct. 5, 1909

JUNIORS (*Continued*).

	Date of Membership.
SCHOBINGER, GEORGE. 181 La Salle St., Chicago, Ill	Oct. 5, 1909
SCOTT, WALTER VANDERBILT. 511 Gifford Pl., Hudson, N. Y.	Nov. 8, 1909
SHANKLAND, RALPH GRAHAM. Supt. of Concrete Constr. for E. C. & R. M. Shankland, 1106 The Rookery Bldg., Chicago, Ill	Nov. 8, 1909
SNYDER, HUNTER IMBODEN. 21 Cotton Exchange Bldg., Memphis, Tenn	Nov. 30, 1909
STEWART, WALTER PHELPS. U. S. Engr. Office, Nashville, Tenn	Nov. 8, 1909
SWENSSON, OTTO JORDAN. Office, Board of Water Supply Yorktown Heights, N. Y	Nov. 8, 1909
TATUM, ROBERT LEE. Benoit, Miss	Nov. 8, 1909
WINANS, LAWRENCE LEWIS. County Surv., Audrain Co., Mexico, Mo	Nov. 8, 1909

RESIGNATION

ASSOCIATE	Date of Resignation.
LOWE, HOUSTON	Nov. 8, 1909

DEATHS

BALL, ERNEST STEARNS. Elected Junior, May 2d, 1905; date of death unknown.

HARRIS, CHARLES MARSHALL. Elected Associate, January 8th, 1873; died October 28th, 1909.

SANDERSON, JAMES GARDNER. Elected Member, July 6th, 1881; died November 15th, 1909.

Total Membership of the Society, December 7th, 1909, 5 273.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(November 9th to December 6th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England, 25c.
(22) *Iron and Coal Trades Review,* London, England, 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $4.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Canada, 15c.
(97) *Turbine*, Berlin, Germany, 1 Mark.
(98) *Journal*, Engrs. Soc. Pa., 219 Market St., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $1.
(100) *Professional Memoirs*, Engr. Bureau, U. S. A., Wash. Barracks, D. C., $1.25.

LIST OF ARTICLES

Bridges.

Computation of Stresses in Open-Webbed Arches without Hinges.* C. W. Hudson. M. Am. Soc. C. E. (54) Vol. 65.

The Sixth Street Viaduct, Kansas City.* E. E. Howard, Assoc. M. Am. Soc. C. E. (54) Vol. 65.

Walnut Lane Bridge, Philadelphia.* George S. Webster, and Henry H. Quimby, Members, Am. Soc. C. E. (54) Vol. 65.

The Pneumatic Founding of Pier No. 6, Banas River Bridge, Nagda-Muttra State Railway.* Victor Bayley, Assoc. M. Inst. C. E. (63) Vol. 77.

Road-Bridges over the Nile at Cairo.* Arthur James Knowles, M. Inst. C. E. (63) Vol. 77.

Report (Great Britain) on the Question of Junctions and Swing Bridges; Elimination of Slacking.* C. L. Morgan. (Subject for discussion at the 8th Inter. Ry. Cong.) (88) Nov.

Strengthening Bridges with a View to Increasing the Speed of Trains. (Report Spain and Portugal).* R. Coderch. (Subject for discussion at the 8th Inter. Ry. Cong.) (88) Nov.

Strengthening the Bridges with a View to Increasing the Speed of Trains. (Report Belgium and Holland).* J. Schroeder van der Kolk. (Subject for discussion at the 8th Inter. Ry. Cong.) (88) Nov.

Special Track Elevation Bridges in Chicago.* (14) Nov. 13.

The Three-Hinged Reinforced-Concrete Arch Across the Vermillion River, Wakeman, O. Wilbur J. Watson. (Abstract of paper read before the Ohio Eng. Soc.) (13) Nov. 18.

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*Illustrated.

Bridges—(Continued).

The Linda Vista Bridge: A Reinforced Concrete Trestle, Pasadena, Cal.* (13) Nov. 18; (14) Nov. 20.
Erecting Vertically a Long Rolling Lift Bridge.* (14) Nov. 20.
The Design of the Broadway or Sparkman St. Bridge, Nashville, Tenn.* Howard M. Jones, M. Am. Soc. C. E. (13) Nov. 25.
Reinforced Concrete Viaduct at Rotterdam.* E. Ommeganck. (15) Nov. 26.
Die Talbrücken der Neubaustrecke Metz-Vigy-Anzelingen.* Storm. (49) Vol. X.-XII., 1909.
Die Gewölbte Brücke über den Isonzofluss bei Saleano.* Rudolf Jaussner. (53) Oct. 29.
Eiserne Eisenbahn-Brückentragwerke mit Wagrechter Auflagerung.* Robert Schönhöfer. (53) Nov. 12.

Electrical.

Features of the Design and Operation of Rolling Mill Motors and Their Accessories.* W. Manktelow. (26) Nov. 5.
The Lepel Wireless Telegraph System.* Baron von Lepel. (73) Nov. 5.
Electric Cranes.* Warren H. Miller. (27) Nov. 11.
A Damped Quadrant Electrometer for Submarine Cable Testing.* R. M. Sayers. (73) Nov. 12; (11) Nov. 12.
No-Load Conditions in Self-Excited, Single-Phase, Shunt-Induction Motors.* Val. A. Fynn. (27) Serial beginning Nov. 18.
Telephone Trials with a Long-Distance Microphone. C. Egnér 1. G. Holmstrom. (73) Nov. 19.
Central-Station System of the Easton Gas and Electric Company.* (27) Nov. 25.
Location of Faults in Underground Cables.* William A. Durgin. (27) Nov. 25.
Steel Towers for Transmission Lines.* E. Kilburn Scott, A. M. Inst. C. E. (22) Nov. 26.
Application of Variable-Speed Three-Phase Motors to the Driving of Machines for Printing Textiles.* (73) Nov. 26.
Telegraphing Pictures.* T. Thorne Baker. (29) Nov. 26.
Electric Heaters. H. M. Phillips. (10) Dec.
Central-Station Electrical Motor Service. Walter Stuart Kelley. (27) Dec. 2.
Generating Station of the Home Electric Light & Steam Heating Company at Tyrone, Pa.* (27) Dec. 2.
Underground Distribution for Small Cities and Towns.* J. Franz. (27) Dec. 2.
Depreciation as an Element of Rate-Making. (27) Dec. 2.
Wireless Telegraphy and Telephony.* H. Marchand. (19) Dec. 4.
Le Congrès International des Applications de l'Electricité et l'Exposition de Marseille. F. Leprince-Ringuet. (43) Sept.
La Nouvelle Usine Electrique de la Compagnie du Chemin de Fer d'Orleans, à Tours.* R. Durand-Perroux. (33) Nov. 13.

Marine.

The Design of Marine Steam-Turbines.* Stanley John Reed, Assoc. M. Inst. C. E. (63) Vol. 177.
Speed Reduction and Reversing with Turbine Driver. (11) Nov. 5.
Propelling Machinery for Warships. H. J. Oram. (Paper read before the Jun. Inst. of Engrs.) (12) Serial beginning Nov. 19; (11) Nov. 19.
The Design of Reversing Engines.* Edward M. Bragg. (95) Serial beginning Dec.
The *Monitoria.** Arthur H. Haver. (95) Dec.
The New Steamship *Wilhelmina.** (95) Dec.
The Marine Polar Motor.* Hugo Anderson. (95) Dec.
The Design of Ships' Rudders.* R. J. Grimshaw. (10) Dec.
Le *Tortuguero*, Vapeur à Cales Réfrigérées, Affecté au Transport des Bananes.* (33) Nov. 20.

Mechanical.

Some Recent Grain-Handling and Storing Appliances at the Millwall Docks.* Magnus Mowat, M. Inst. C. E. (63) Vol. 77.
The Effect of Varying the Degree of Saturation of the Air-Supply to a Suction-Gas Producer.* Arnold Hartley Gibson, Assoc. M. Inst. C. E. and Reginald Duncan Gwyther, Stud. Inst. C. E. (63) Vol. 77.
The Road-Motors of the Present Day, and Some Unsolved Problems Connected with Them. Henry Capel Lofft Holden. (63) Vol. 77.
Metal Planing Machines. Geo. H. Benzon, Jr. (2) July.
The Cleansing of Clay.* Richard Hoffmann. (76) Nov.
A New Type of Water Tube Boiler.* T. H. McGraw, Jr. (58) Nov.
Vertical Retorts at St. Helens.* S. Glover. (Paper read before the Manchester Dist. Inst. of Gas Engrs.) (66) Nov. 2.
Some Investigations with a New Form of Torsion Meter.* (12) Nov. 5.

*Illustrated.

Mechanical—(Continued).

The Schulz Steam Turbine.* (12) Nov. 5.
Aeroplane Frames.* (11) Nov. 5.
The Development of Centrifugal Pumps. J. D. Berg. (From paper read before the Central States W. W. Assoc.) (13) Nov. 11.
An Oil-Engine Central Station.* (27) Nov. 11.
The Oxyhydric Process of Cutting and Welding Metals; its Use Following the Pabst Boiler Explosion.* (20) Nov. 11.
The Panama Canal Cableways.* (20) Nov. 11; (86) Nov. 17; (96) Dec. 3.
Stresses Due to Bending and Twisting and the Design of Shafting.* A. Lewis Jenkins. (11) Nov. 12.
Brown-Boveri Turbine Plant at Buenos Ayres.* (26) Nov. 12.
Tests of a Small Steam-Turbine Plant.* George W. Martin. (27) Nov. 18.
The Mechanism of Cotton-Spinning Machinery.* W. H. Cook. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Nov. 19.
Motor Car Exhibition at Olympia.* (12) Serial beginning Nov. 19; (11) Serial beginning Nov. 19.
The Farman Biplane.* (19) Nov. 20.
The Rock Crushing Plant at Porto Bello for the Panama Canal.* (14) Nov. 20.
Public Service Corporation Power Plant.* Warren O. Rogers. (64) Nov. 23.
An Interesting Aerial Cableway and Orehandling Plant in New Caledonia.* (19) Nov. 27.
The Exhaust Steam Turbine.* J. N. Bailey. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Nov. 26.
The Work of an Iron Foundry Laboratory.* George Hailstone. (Paper read before the Coventry Eng. Soc.) (47) Serial beginning Nov. 26.
The Schenck Bucket Conveyor.* (22) Nov. 26.
The McMyler Coal Hoist.* (22) Nov. 26.
Oil Handling in Brooklyn.* (17) Nov. 27.
Gas Distribution in San Francisco.* W. R. Morgan. (Paper read before the Pacific Coast Gas Assoc.) (24) Nov. 29.
Plant of the Hartford Electric Light Company.* (64) Nov. 30.
The Cableway and its Uses.* Sterling H. Bunnell. (10) Dec.
The Development of the Aeroplane.* Henry Harrison Suplee. (10) Serial beginning Dec.
The Frankfort Aeronautical Exhibition.* J. O. Newman. (10) Dec.
Cranes and Hoisting Machinery.* J. E. Springer. (10) Serial beginning Dec.
The Evolution of the Planing Machine.* Thomas R. Shaw. (10) Serial beginning Dec.
Gas-Engine and Producer Installations for Manufacturing Plants.* Percival Robert Moses. (9) Dec.
Refrigeration and Electricity Generating Stations. Sydney F. Walker. (27) Dec. 2.
A Novel Locomotive Rock Drill and a Remarkable Drilling Record.* Frank Richards. (13) Dec. 2.
An Unique Direct-Acting Explosion Pump.* (13) Dec. 2.
Tool Lay-Out for the Scranton Shops of the Delaware, Lackawanna & Western. (15) Dec. 3.
A Novel Aerial Railway.* (46) Dec. 4.
Machines à Vapeur étude sur les Machines à Tiroirs Pistons.* E. Lefer. (37) Oct. 31.
Remarques sur le Fonctionnement des Pompes Centrifuges.* R. Tison. (37) Oct. 31.
Les Pare-Boue pour Autobus, le Concours de la Compagnie Gle. des Omnibus de Paris.* E. Bret. (33) Nov. 20.
Allgemeines über Walzlinie und Oberdruck.* J. Puppe. (50) Oct. 27.
Einschalt-, Anlauf- und Ausschaltperiode der Druckwasserhebemaschinen beim Betriebe durch Gewichtakkumulatoren. Karl Meyer. (53) Oct. 29.
Der Kolbenkompressor und der Turbokompressor. J. Havlicek. (48) Oct. 30.
Ueber den Einfluss der Flüchtigen Bestandteile Fester Brennstoffe auf dem Wirkungsgrad von Kesselanlagen mit Innenfeuerung.* E. J. Constam and P. Schlapfer. (48) Serial beginning Nov. 6.
Die Geschichte des Modernen Kugellagers.* P. Bruhl. (48) Serial beginning Nov. 6.
Eisenbetonschornsteine.* H. Weidner. (Paper read before the Verein Deutscher Firmen für Schornsteinbau und Feuerungsanlagen.) (80) Nov. 16.
Der Maschinelle Betrieb auf Hüttenwerken. E. Riecke. (50) Nov. 24.

Metallurgical.

Historic Old Plant of the Dunbar Furnace Company.* (62) Nov.
Aluminium-Copper-Tin Alloys.* C. A. Edwards and J. H. Andrew. (Paper read before the Inst. of Metals.) (11) Nov. 12.
Notes on Malleable Cast Iron. Rudolf Namias.* (Tr. abstract of paper read before the Inter. Cong. of Applied Chem.) (11) Nov. 12.

*Illustrated.

Metallurgical—(Continued).

"Slag Inclosures" in Steel.* Walter Rosenhain. (Paper read before the Inter. Assoc. for Testing Materials.) (20) Nov. 18.
Sparks as Indications of Different Kinds of Steel.* Max Berman. (Abstract of paper read before the Inter. Assoc. for Testing Materials.) (47) Nov. 19.
The Baghouse and its Recent Applications. W. C. Ebaugh. (Abstract from *Jour. Ind. and Eng. Chem.*) (16) Nov. 20.
Wooden Jig Grates in the Joplin District.* Otto Ruhl and Frank Sansom. (16) Nov. 20.
The New York State Steel Company's Plant.* (20) Nov. 25.
Treated Steel for High Service.* Jas. H. Baker. (20) Nov. 25.
The Roosevelt Deep Drainage Tunnel, Colorado.* Rufus M. Bagg, Jr. (16) Nov. 27.
The Fuel Economy of Dry Blast.* R. S. Moore. (45) Dec.
The World's Largest Chimney.* R. L. Herrick. (45) Dec.
The Electrochemical System of Amalgamation and Cyanidation. Elmer Ellsworth Carey. (9) Dec.
Altoona Foundries, Pennsylvania Railroad.* (15) Dec. 3.
Electric Smelting with the Girod Furnace.* W. Borchers. (16) Dec. 4.
Traitement de l'Acier au Four Electrique. Henry M. Howe. (Tr. by Henri Marelle.) (93) Nov.
Magnetstahl und Permanenter Magnetismus.* G. Mars. (50) Serial beginning Oct. 27.
Der Girod-Ofen und die Elektrischen Schmelzwerke System Paul Girod. W. Borchers. (50) Nov. 10.
Ueber Elektrische Öfen mit Besonderer Berücksichtigung der Elektrostahldarstellung.* Viktor Engelhardt. (53) Serial beginning Nov. 19.

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The Sinking of Circular Shafts.* Robert Steven. (59) Vol. 32, Pt. 1.
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Der Bau Zweiter Schleusen bei Wernsdorf und Kersdorf. (Spree-Oder-Wasserstrasse.)* Engelhard u. Zimmermann. (49) Vol. X.-XII., 1909.

*Illustrated.

REPORTS OF SPECIAL COMMITTEES.

Progress Report of the Special Committee on Bituminous Materials for Road Construction.

The Board of Direction,
American Society of Civil Engineers:

GENTLEMEN:

Your Special Committee, appointed May 4th, 1909, to report on "Bituminous Materials for Road Construction and on Standards for Their Test and Use," respectfully presents the following Progress Report.

On receiving notice of its appointment, your Committee organized by electing W. W. Crosby, Chairman, and A. H. Blanchard, Secretary. Your Committee agreed at once that the available information concerning its subject was so diversified in both form and character as to render it impossible of satisfactory use by the Committee, and the question of securing additional information on uniform lines was considered.

As actual construction had already begun in many instances, it seemed imperative to establish at the earliest possible moment a schedule of information concerning such work, which should cover all the points on which it appeared that information was necessary, and yet which should, if possible, not be so extensive in form as to deter anyone from attempting to respond to a large portion of the questions asked.

In preparing this schedule, the earlier reports from many sources were considered, and, as far as seemed to the Committee practicable, an effort was made to include in its schedule the points that seemed to others important. It was also found necessary, in order to secure uniformity, to suggest methods for arriving at some of the figures requested. As far as it seemed practicable, already standardized methods were adopted and only such changes in ordinary methods were inserted as seemed necessary for accuracy or improvement. In some instances new methods had to be devised.

The Committee submitted a list of analyses and methods of testing bituminous materials to many engineers and chemists, conversant with the subject, for criticism and suggestions. After considerable correspondence, and a series of meetings of the Committee, and after continued revision, the final list was adopted as representing most nearly the views of the majority.

A circular letter was then prepared, explaining the intentions of the Committee and inviting the co-operation and assistance of those interested in its work.

The letter and forms were printed and distributed about August 1st to National, State, County, Town and City Highway Engineers and Officials, and to Chemists, Manufacturers of, and Agents for, Bituminous Materials. More than 800 letters were sent out during August and September.

From replies already received, your Committee feels greatly encouraged by the interest manifested in its work, in spite of, as we recognize, the amount of work requested by it of its correspondents. The States of Maine, Massachusetts, Maryland, Pennsylvania, New Hampshire, New York, and Rhode Island have agreed to report to the Committee on the bituminous macadam roads constructed during 1909, including the analyses of the bituminous materials used, made in accordance with the methods suggested by the Committee.

At this date, your Committee can scarcely report more than "Progress." Conclusions cannot be drawn on most points until the results of the action of the varying seasons and traffic conditions shall have become apparent. It is hoped that partial conclusions may be justified in time to report at the next Annual Meeting.

The Committee begs leave to express here its profound appreciation of the courtesy and material assistance rendered it in its work by your Secretary, Mr. Charles Warren Hunt, and also by the engineers, chemists, and manufacturers with whom it has had the good fortune to come in contact, and to sign itself

Very respectfully,

W. W. Crosby, *Chairman.*
H. K. Bishop.
A. W. Dean.
A. H. Blanchard, *Secretary.*

Progress Report of the Special Committee on Steel Columns and Struts.

To the Members of the American Society of Civil Engineers in Annual Meeting:

The Special Committee authorized by vote of this Society "to consider and report upon the design, ultimate strength and safe working values of Steel Columns and Struts" submits the following report of progress.

Your Committee met and organized on January 6th, 1909. Since that time four meetings of the Committee as a whole and numerous sub-committee meetings have been held. Time and labor have been spent in collecting existing literature on the subject of tests of compression members, in investigating and compiling the data now accessible, in a preliminary consideration of the extent to which it is safe to draw conclusions from data now available, and in a study of the possibility of securing additional information. At the last Annual Meeting, your Committee submitted to the Society a resolution requesting the United States Government to construct a machine capable of testing to destruction compression members of large dimensions.

Your Committee finds that there are at present available for use in this country, but three testing machines of large size, while only two are capable of making compression tests. The data giving principal dimensions and capacity of the machines are submitted in the following table:

DATA RELATING TO TESTING MACHINES.

	Phœnixville, Pa.	Ambridge, Pa.	Watertown, Mass.
Date of completion	About 1886	1905	1879
Capacity for tension tests	2 600 000 lb.	4 000 000 lb.	500 000 lb.
Capacity for compression tests.	2 600 000 lb.	none	800 000 lb.
Maximum length of test pieces:			
Tension	50 ft.	39 ft.	20 ft.
Compression	55 "	none	26 " 3 in.
Available clear width for tests.	3 "	3 ft.	42¼ in.
Power	Hydraulic	Hydraulic	Hydraulic
Recording device	Mercury gauges	Mercury gauge	Emery hydraulic scale
Height from floor to center of ram	About 4 ft.	2 ft.	47 in.

These machines will be supplemented in the near future by two others. A vertical machine of 10 000 000 lb. capacity in compression, capable of testing columns up to 65 ft. in length is now being built for the Geological Survey. This machine will be located in Pittsburg. Under authority of the Congress of 1908-09, the Bureau of Standards has contracted for a horizontal machine with a capacity of 2 300 000 lb.

in compression and 1 150 000 lb. in tension, of the same type that is now in use at Watertown Arsenal. This machine is to be located at Washington, D. C.

Your Committee has examined the records of numerous compression tests made on pieces varying in area of cross-section from laboratory specimens to 42 sq. in. While the results given by small specimen tests are interesting, your Committee has selected only the tests of columns large enough to be used as structural members, and these have been plotted on Figs. 1 to 22, inclusive. Fig. 23 gives a summary of results.* Your Committee has also considered some tests which it is not now at liberty to publish, and it is expected that the data given in this report will be supplemented from time to time as these and other tests become available for publication.

In the use of $\frac{l}{r}$ (length divided by radius of gyration) as a variable, your Committee realizes that r may not be the only variable affecting the strength of a column of given length. Shape, details, and dimensions, may make the results very different in columns having the same value of $\frac{l}{r}$. Until many extended series of tests afford more information with reference to these elements, it is necessary to follow common practice in plotting.

To reduce the results of the various series of tests shown on the diagrams to a uniform basis, the values have been adjusted to a grade of steel of 60 000 lb. ultimate tensile strength in specimen tests, by assuming that the ultimate compressive strength will vary as the ultimate tensile strength. This method of adjustment probably does not accurately represent the law of variation of the different grades of iron and steel, still, as the adjusted value is a fair medium between the older iron and the newer steel, it affords some basis for comparison.

A casual examination of the plates shows a meager range of experiments, made on no systematic plan, and a striking disagreement of results, even after adjustment for quality of material. It is evident that these data are not sufficient for the establishment of a satisfactory law for the strength of columns. This emphasizes the importance of further tests on columns arranged in series under conditions of material, workmanship and details, which will aid scientific observation in the establishment of the effects of the different variables.

At the time your Committee was appointed, circumstances caused a demand for tests to destruction of compression members equal in size to the largest ever built for existing bridges. This demand was backed largely by both the popular and technical press, but careful consideration and study lead to the conviction that while such tests are desirable,

*Not received in time to be reproduced with this report.

the cost of making them is so great that there is no chance of obtaining a sufficient number to disclose any fundamental law. Elaborate series of tests on smaller columns, in which the details and processes of manufacture conform to commercial practice, would disclose the effects of the different variables and would determine the fundamental laws of the behavior of columns under stress. Such a series of tests, with the necessary investigations, should properly be made by the Federal Government. For comparison with the results obtained from these series, tests of members, modeled after columns which are too expensive to allow more than an occasional test, and too large to be handled conveniently, would afford valuable data.

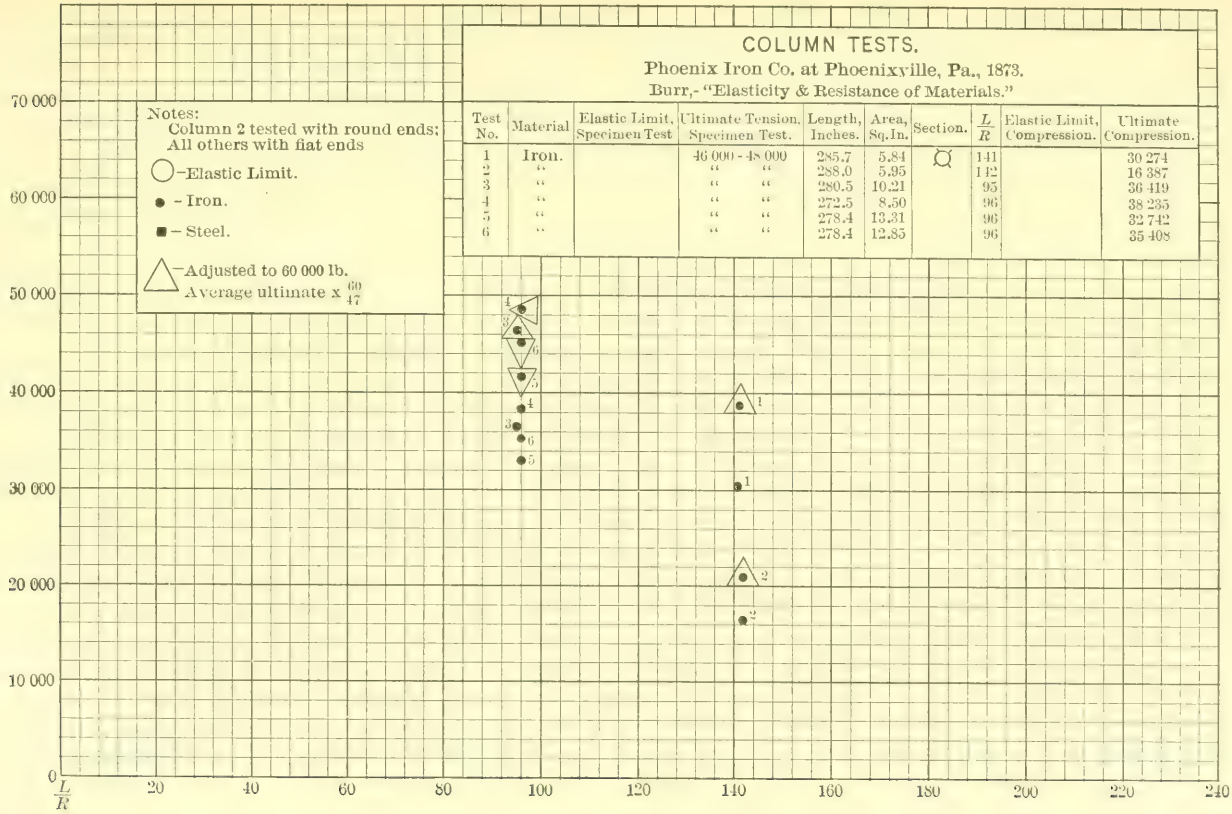
Your Committee recognizes the urgency of the demand for a statement giving values for ultimate strength and safe working stresses for compression members, but in their opinion existing data do not warrant the recommendation of any definite column formula. Until the subject can be studied more at length and until more extended series of tests have been made, your Committee does not deem it advisable to complicate the situation by offering a tentative formula which would vary but little from other formulas now in use. When the results of further tests become available for study and investigation, your Committee hopes to be able to make a recommendation for a column formula.

Committee.

Alfred P. Boller.
Austin Lord Bowman.
Emil Gerber.
Charles F. Loweth.
Ralph Modjeski.
Frank C. Osborn.
Geo. H. Pegram.
Lewis D. Rights.
Geo. F. Swain.
Emil Swensson.
Joseph R. Worcester.

For the Committee,

Austin Lord Bowman,
Chairman.
Lewis D. Rights,
Secretary.



Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	L/R	Elastic Limit, Compression.	Ultimate Compression.
1	Iron.		46 000 - 48 000	285.7	5.84	○	141		30 274
2	"		" "	288.0	5.95		142		16 387
3	"		" "	280.5	10.21		95		36 419
4	"		" "	272.5	8.50		96		38 235
5	"		" "	278.4	15.31		96		32 742
6	"		" "	278.4	12.85		96		35 408

FIG. 1.

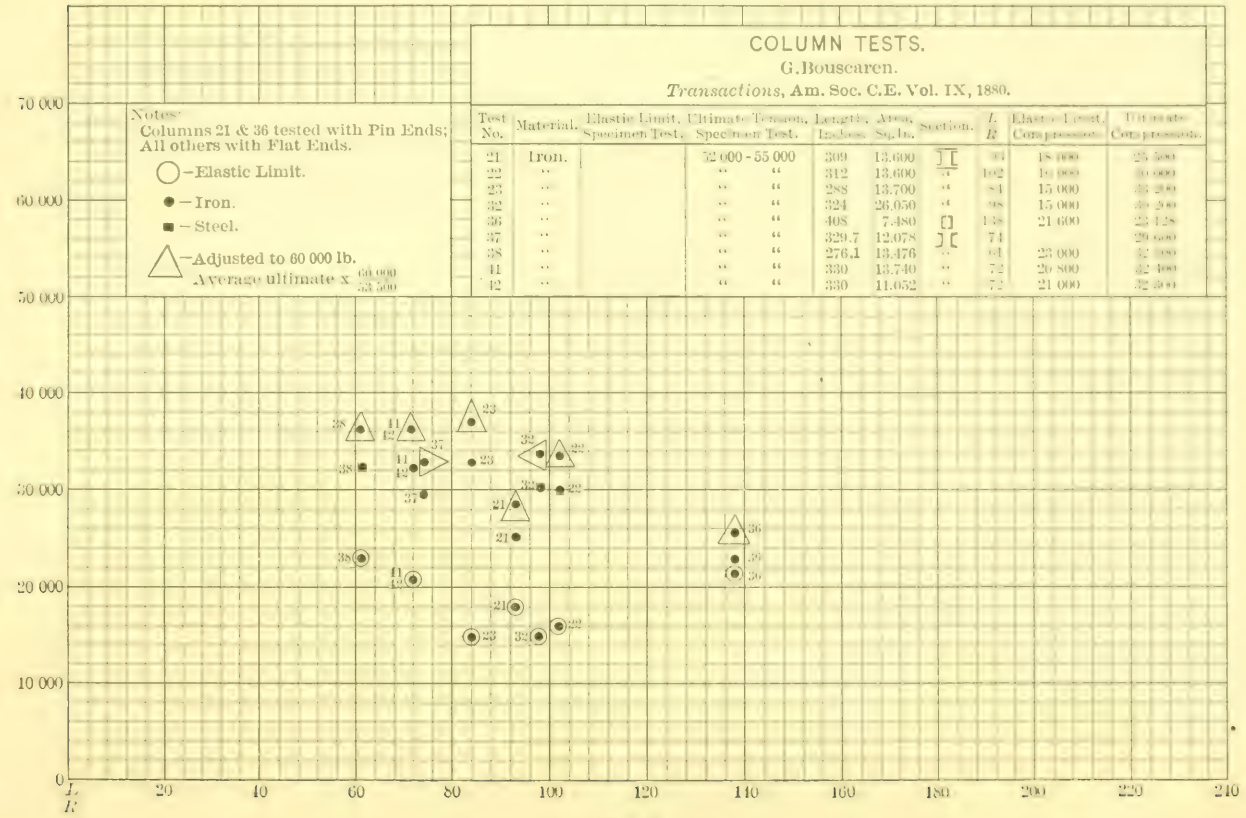
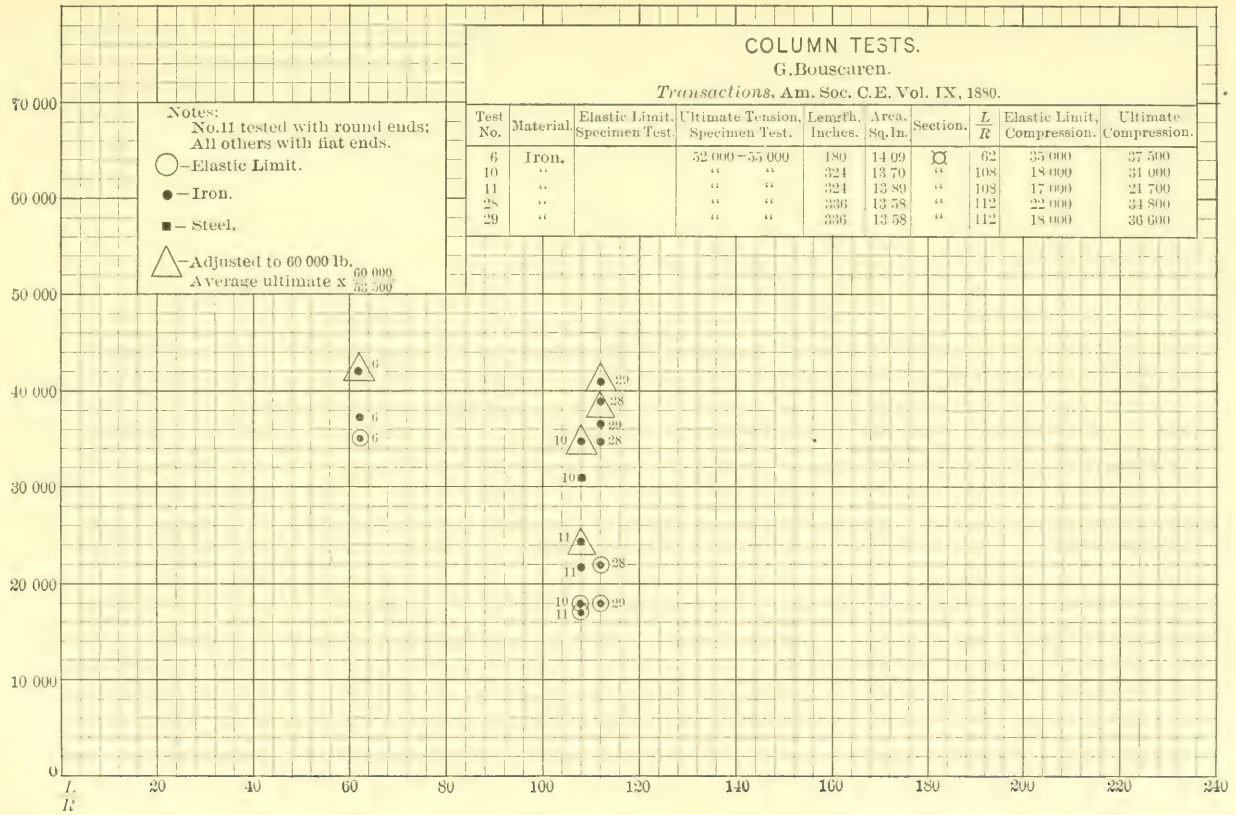

COLUMN TESTS.
G. Bouscaren.
Transactions, Am. Soc. C.E. Vol. IX, 1880.
Notes:
Columns 21 & 36 tested with Pin Ends;
All others with Flat Ends.
○ – Elastic Limit.
● – Iron.
■ – Steel.
△ – Adjusted to 60 000 lb.
Average ultimate x 60 000 / 53 500
Test No. | Material. | Elastic Limit, Specimen Test. | Ultimate Tension, Specimen Test. | Length, Inches. | Area, Sq. In. | Section. | L/R | Elastic Limit, Compression. | Ultimate Compression.
21 | Iron. | | 52 000 - 55 000 | 309 | 13.600 |][| 84 | 18 000 | 25 500
42 | " | | " " | 312 | 13.600 | " | 102 | 14 000 | 21 000
23 | " | | " " | 288 | 13.700 | " | 84 | 15 000 | 23 250
32 | " | | " " | 324 | 26.050 | " | 98 | 15 000 | 23 250
36 | " | | " " | 408 | 7.480 | [] | 138 | 21 600 | 22 428
37 | " | | " " | 329.7 | 12.078 |][| 74 | | 20 600
38 | " | | " " | 276.1 | 13.476 | " | 64 | 23 000 | 32 000
41 | " | | " " | 330 | 13.740 | " | 72 | 20 800 | 32 400
42 | " | | " " | 330 | 11.052 | " | 72 | 21 000 | 32 500
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 2.



Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
6	Iron.		52 000–55 000	180	14 09	⌻	62	35 000	37 500
10	"		" "	324	13 70	"	108	18 000	31 000
11	"		" "	324	13 89	"	108	17 000	21 700
28	"		" "	336	13 58	"	112	22 000	34 800
29	"		" "	336	13 58	"	112	18 000	36 600

FIG. 3.

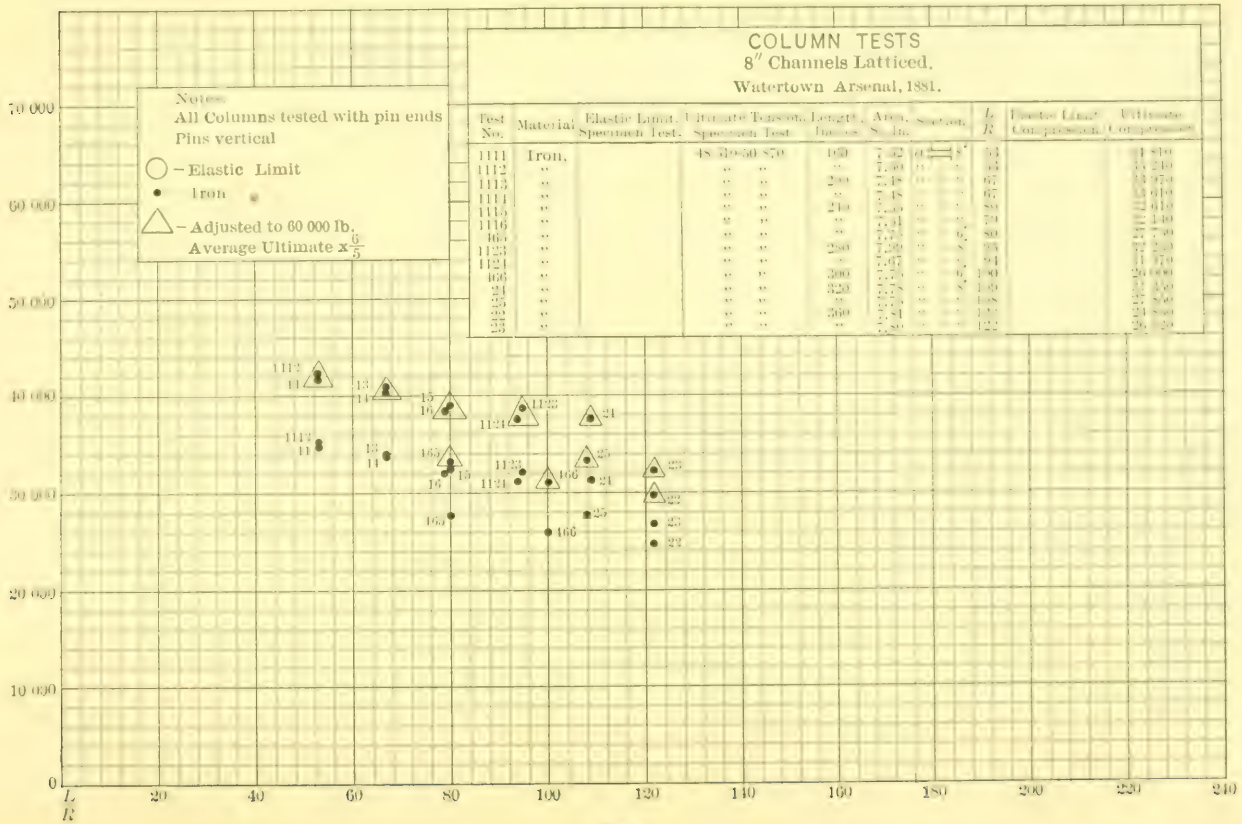

COLUMN TESTS
8″ Channels Latticed.
Watertown Arsenal, 1881.
Notes.
All Columns tested with pin ends
Pins vertical
○ – Elastic Limit
• Iron
△ – Adjusted to 60 000 lb.
Average Ultimate × 6/5
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 4.

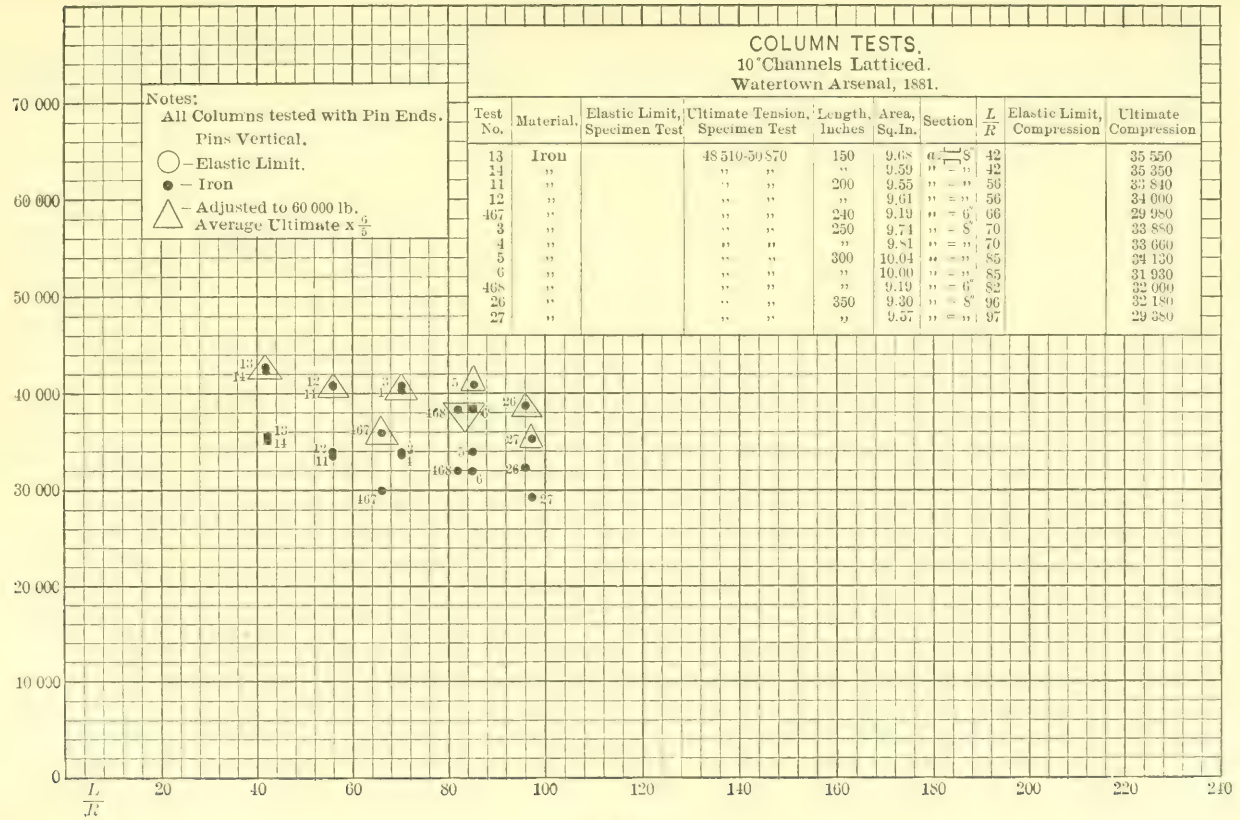

COLUMN TESTS.
10" Channels Latticed.
Watertown Arsenal, 1881.
Notes:
All Columns tested with Pin Ends.
Pins Vertical.
○ – Elastic Limit.
● – Iron
△ – Adjusted to 60 000 lb.
Average Ultimate x 6/5
Test No. | Material. | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length, Inches | Area, Sq.In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression
13 | Iron | | 48 510-50 870 | 150 | 9.68 | a, [I] S | 42 | | 35 550
14 | " | | " " | " | 9.59 | " = " | 42 | | 35 350
11 | " | | " " | 200 | 9.55 | " = " | 56 | | 33 840
12 | " | | " " | " | 9.61 | " = " | 56 | | 34 000
467 | " | | " " | 240 | 9.19 | " = 6" | 66 | | 29 980
3 | " | | " " | 250 | 9.74 | " = 8" | 70 | | 33 880
4 | " | | " " | " | 9.81 | " = " | 70 | | 33 660
5 | " | | " " | 300 | 10.04 | " = " | 85 | | 34 130
6 | " | | " " | " | 10.00 | " = " | 85 | | 31 930
468 | " | | " " | " | 9.19 | " = 6" | 82 | | 32 000
26 | " | | " " | 350 | 9.30 | " = 8" | 96 | | 32 180
27 | " | | " " | " | 9.57 | " = " | 97 | | 29 380
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 5.

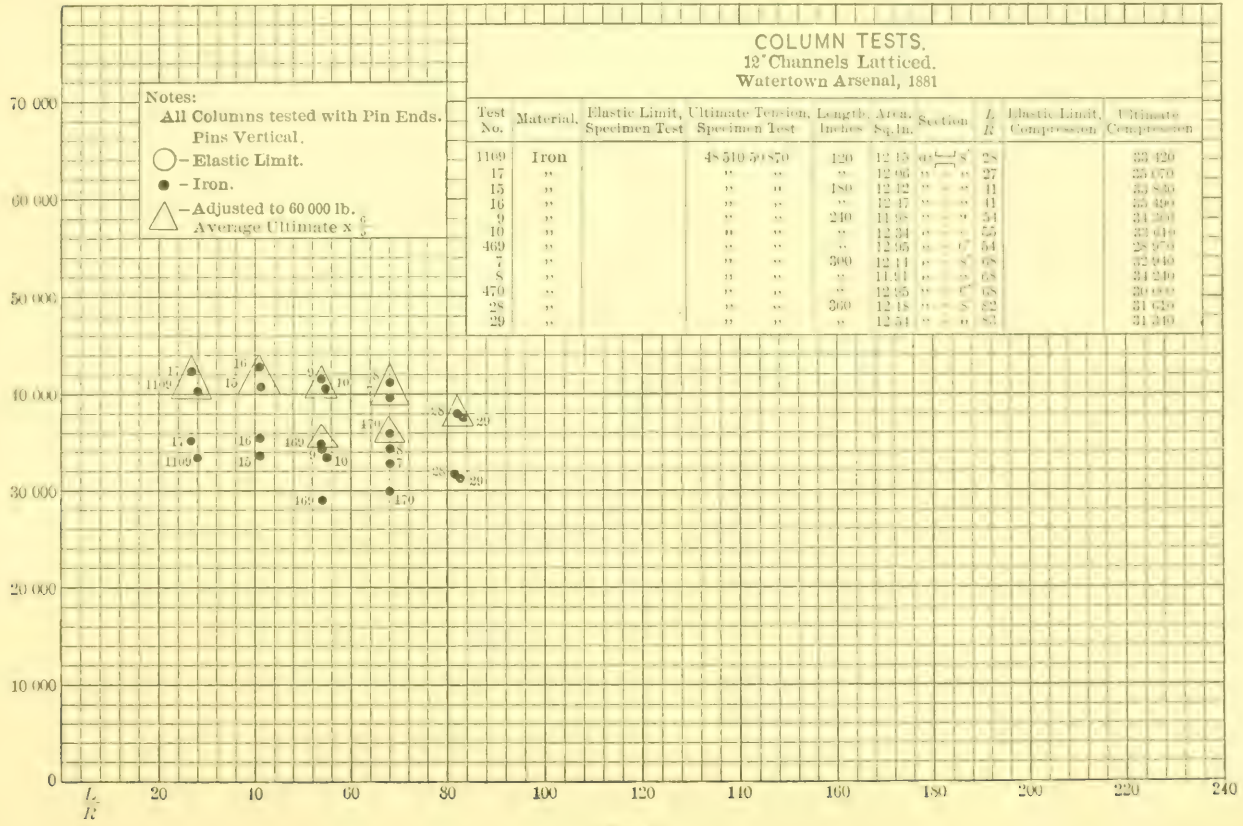
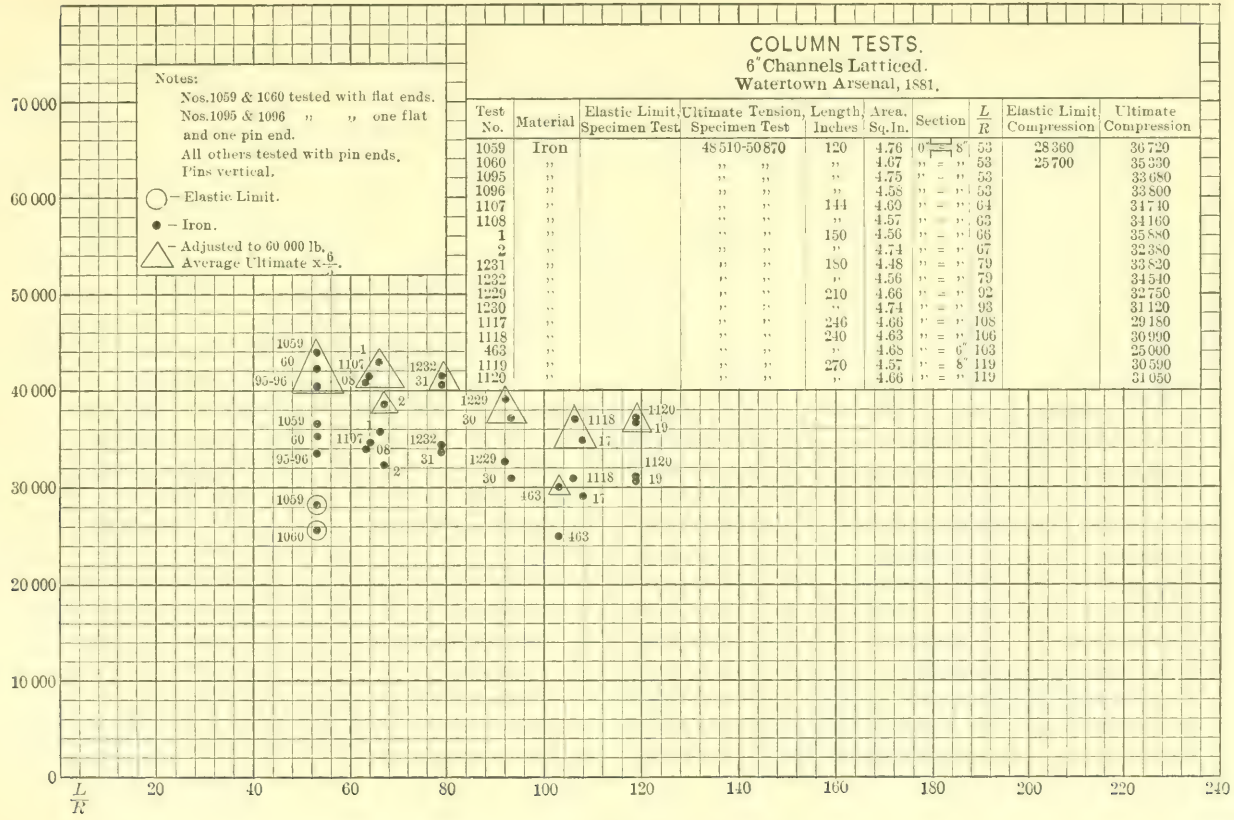

COLUMN TESTS.
12" Channels Latticed.
Watertown Arsenal, 1881
Notes:
All Columns tested with Pin Ends.
Pins Vertical.
○ – Elastic Limit.
● – Iron.
△ – Adjusted to 60 000 lb.
Average Ultimate x 6/5
Test No. | Material. | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length, Inches | Area, Sq. In. | Section | l/r | Elastic Limit, Compression | Ultimate Compression
1109 | Iron | | 48 510 50 870 | 120 | 12 15 | 12" [8" | 28 | | 33 420
17 | " | | " " | " | 12 06 | " " " | 27 | | 35 070
15 | " | | " " | 180 | 12 12 | " " " | 41 | | 33 840
16 | " | | " " | " | 12 47 | " " " | 43 | | 35 490
9 | " | | " " | 240 | 11 98 | " " " | 54 | | 34 260
10 | " | | " " | " | 12 34 | " " " | 55 | | 33 030
469 | " | | " " | " | 12 95 | " " 1/2" | 54 | | 28 970
7 | " | | " " | 300 | 12 14 | " " 5/16" | 68 | | 32 940
8 | " | | " " | " | 11 91 | " " " | 68 | | 34 240
470 | " | | " " | " | 12 95 | " " 5/16" | 68 | | 30 030
28 | " | | " " | 360 | 12 18 | " " 5/16" | 82 | | 31 620
29 | " | | " " | " | 12 54 | " " " | 83 | | 31 240
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
l/r
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 6.



Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
1059	Iron		48 510-50 870	120	4.76	6" = 8"	53	28 360	36 720
1060	"		" "	"	4.67	" = "	53	25 700	35 930
1095	"		" "	"	4.75	" = "	53		33 680
1096	"		" "	"	4.58	" = "	53		33 800
1107	"		" "	144	4.69	" = "	64		34 740
1108	"		" "	"	4.57	" = "	63		34 160
1	"		" "	150	4.56	" = "	66		35 880
2	"		" "	"	4.74	" = "	67		32 380
1231	"		" "	180	4.48	" = "	79		33 820
1232	"		" "	"	4.56	" = "	79		34 540
1229	"		" "	210	4.66	" = "	92		32 750
1230	"		" "	"	4.74	" = "	93		31 120
1117	"		" "	246	4.66	" = "	108		29 180
1118	"		" "	240	4.63	" = "	106		30 990
463	"		" "	"	4.68	" = 6"	103		25 000
1119	"		" "	270	4.57	" = 8"	119		30 590
1120	"		" "	"	4.66	" = "	119		31 050

FIG. 7.

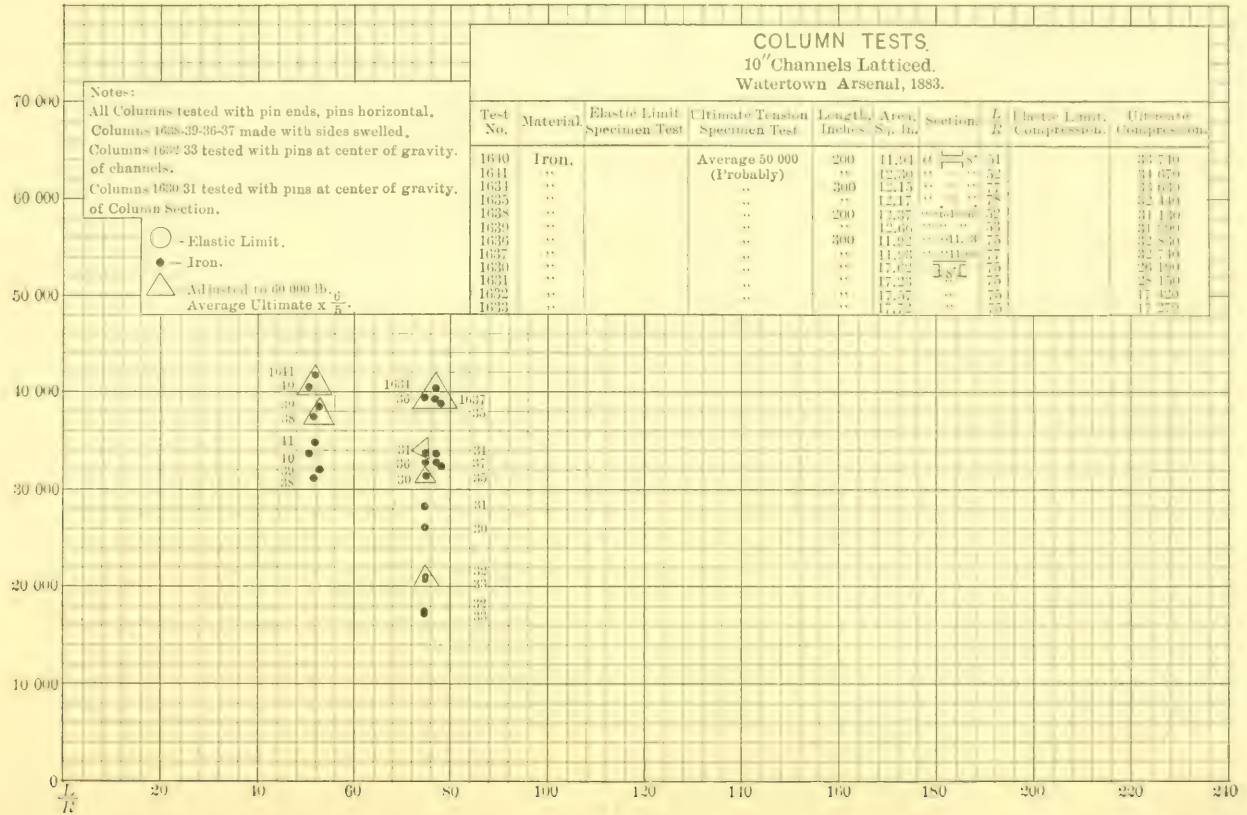

COLUMN TESTS.
10″ Channels Latticed.
Watertown Arsenal, 1883.
Notes:
All Columns tested with pin ends, pins horizontal.
Columns 1628-29-36-37 made with sides swelled.
Columns 1632-33 tested with pins at center of gravity of channels.
Columns 1630-31 tested with pins at center of gravity of Column Section.
○ - Elastic Limit.
● — Iron.
△ Adjusted to 50 000 lb. Average Ultimate x 6/5.
Test No. | Material. | Elastic Limit Specimen Test. | Ultimate Tension Specimen Test. | Length, Inches. | Area, Sq. In. | Section. | L/R | Elastic Limit, Compression. | Ultimate Compression.
1640 | Iron. | | Average 50 000 (Probably) | 200 | 11.94 | 6 | 51 | | 33 740
1641 | " | | | | 12.30 | | 52 | | 34 670
1634 | " | | | 300 | 12.15 | | 78 | | 23 670
1635 | " | | | | 12.17 | | 78 | | 32 440
1628 | " | | | 200 | 12.37 | | 52 | | 31 130
1639 | " | | | | 12.46 | | 52 | | 31 590
1636 | " | | | 300 | 11.92 | 41.3 | 78 | | 32 850
1637 | " | | | | 11.86 | | 77 | | 32 740
1630 | " | | | | 17.62 | | 78 | | 26 190
1631 | " | | | | 17.23 | | 78 | | 28 150
1632 | " | | | | 17.37 | | 78 | | 17 420
1633 | " | | | | 17.72 | | 78 | | 17 270
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 8.

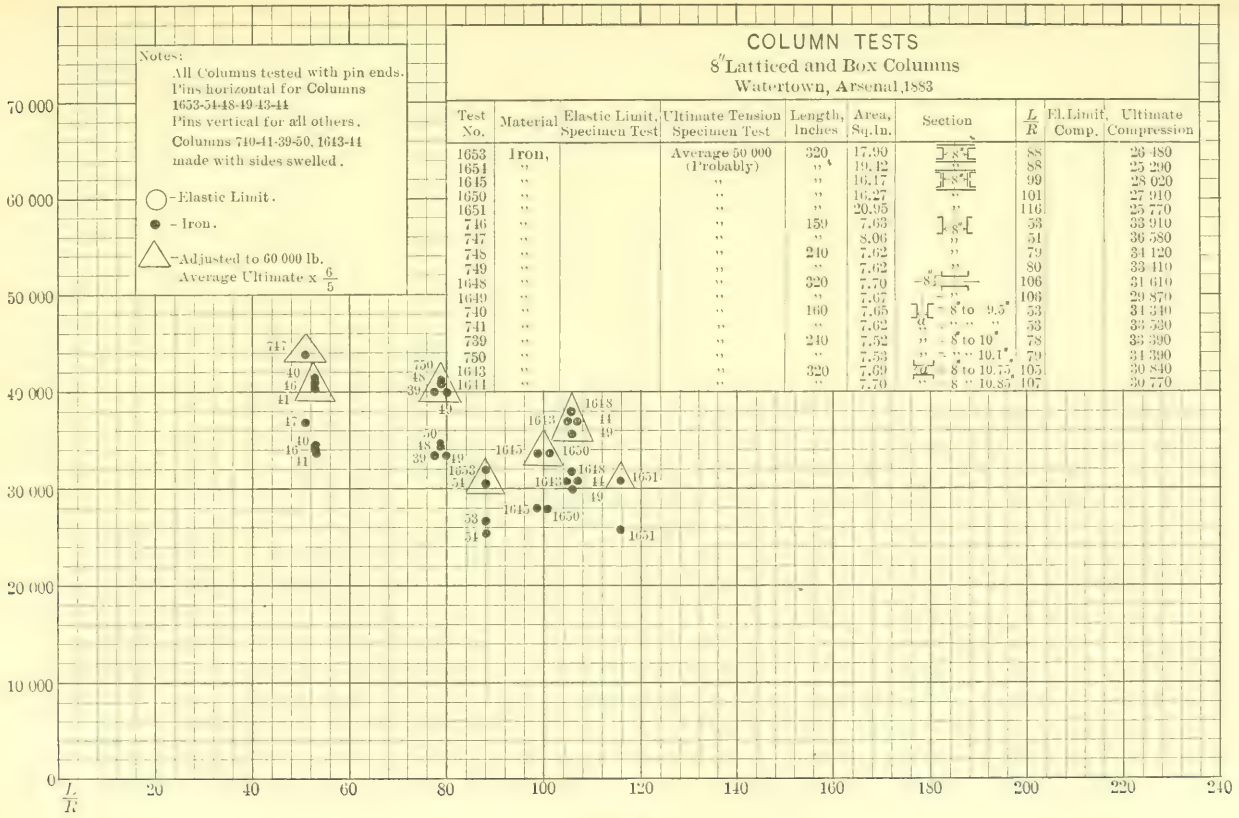
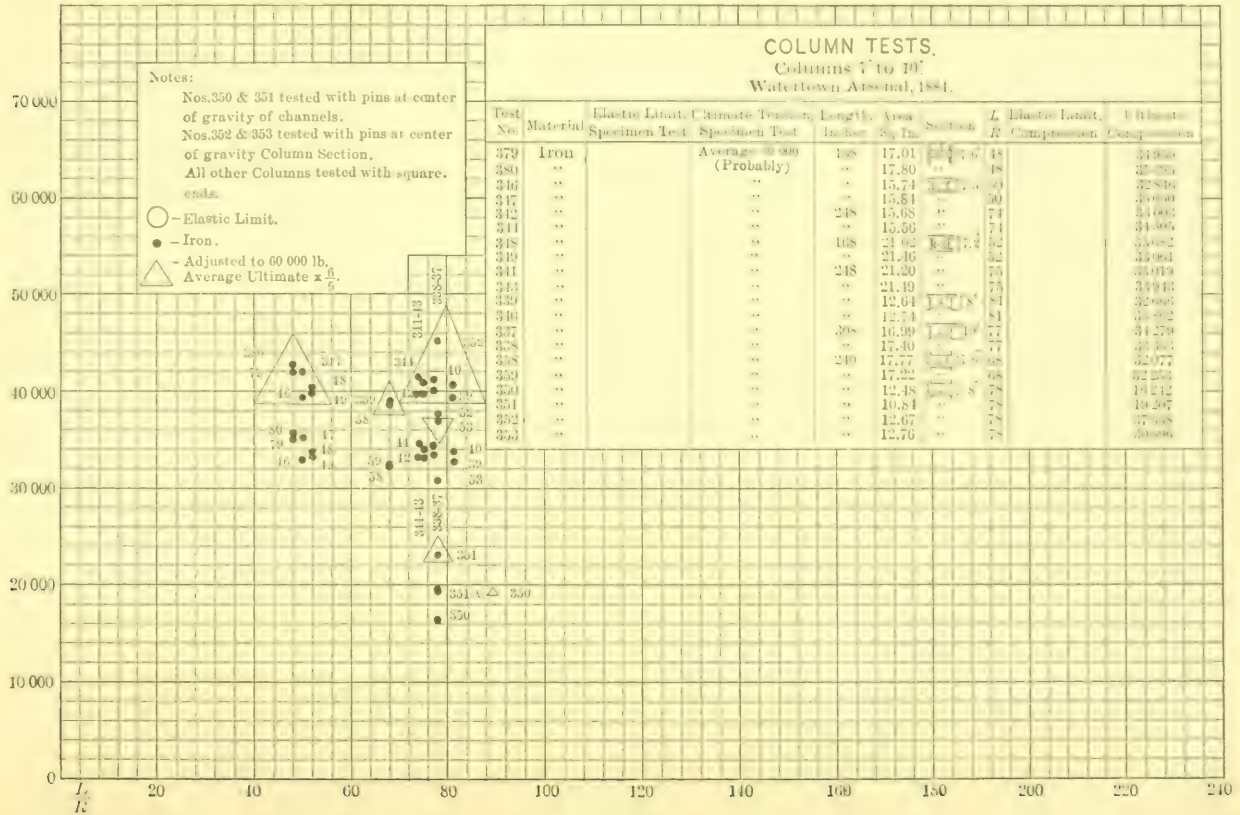
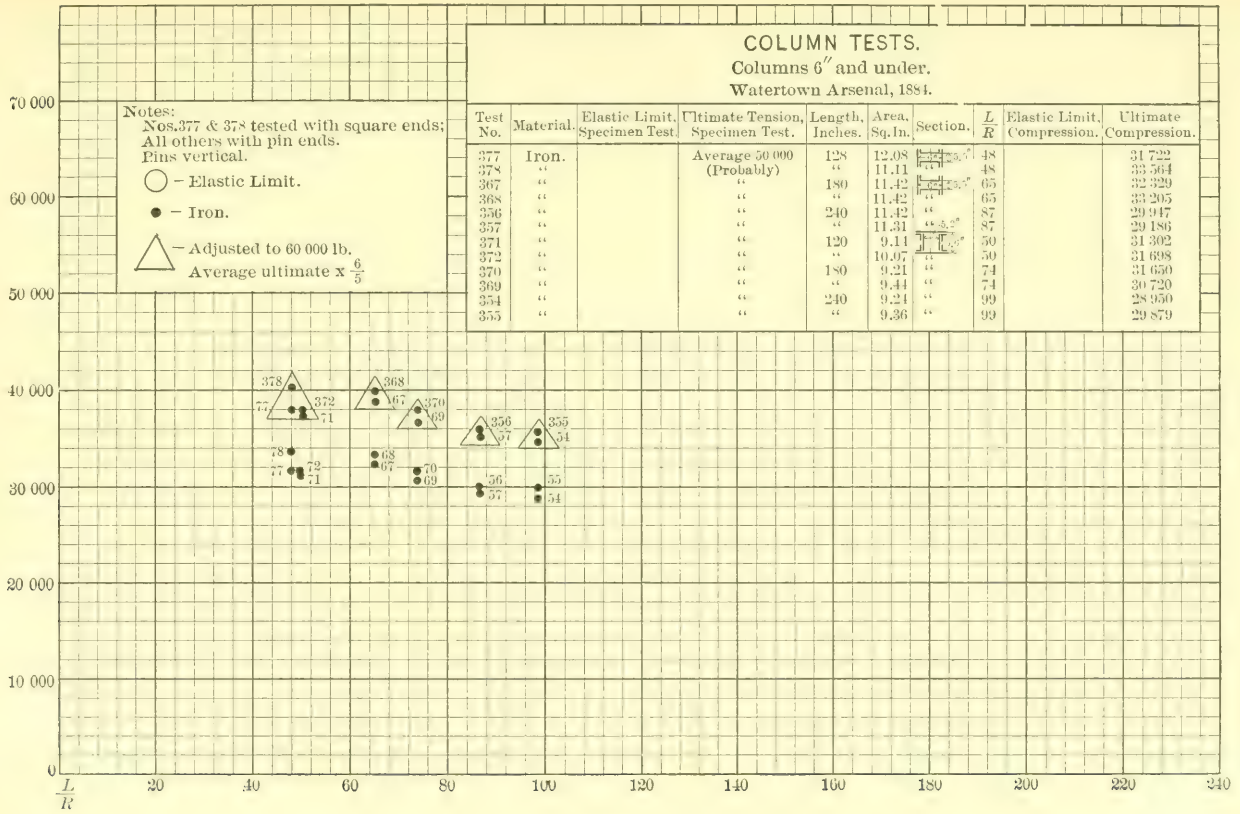

COLUMN TESTS
8″ Latticed and Box Columns
Watertown, Arsenal, 1883
Notes:
All Columns tested with pin ends.
Pins horizontal for Columns
1653-54-48-49-43-44
Pins vertical for all others.
Columns 740-41-39-50, 1643-44
made with sides swelled.
○ - Elastic Limit.
● - Iron.
△ - Adjusted to 60 000 lb.
Average Ultimate x 6/5
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension Specimen Test | Length, Inches | Area, Sq. In. | Section | L/R | Fl. Limit, Comp. | Ultimate Compression
1653 | Iron, | | Average 50 000 (Probably) | 320 | 17.90 | 8″ | 88 | | 26 480
1654 | ,, | | ,, | ,, | 19.42 | 8″ | 88 | | 25 290
1645 | ,, | | ,, | ,, | 16.17 | 8″ | 99 | | 28 020
1650 | ,, | | ,, | ,, | 16.27 | ,, | 101 | | 27 910
1651 | ,, | | ,, | ,, | 20.95 | ,, | 116 | | 25 770
746 | ,, | | ,, | 150 | 7.63 | 8″ | 53 | | 33 910
747 | ,, | | ,, | ,, | 8.06 | ,, | 51 | | 36 580
748 | ,, | | ,, | 240 | 7.62 | ,, | 79 | | 34 120
749 | ,, | | ,, | ,, | 7.62 | ,, | 80 | | 33 410
1648 | ,, | | ,, | 320 | 7.70 | 8″ | 106 | | 31 610
1649 | ,, | | ,, | ,, | 7.67 | ,, | 106 | | 29 870
740 | ,, | | ,, | 160 | 7.65 | 8″ to 9.5″ | 53 | | 34 340
741 | ,, | | ,, | ,, | 7.62 | ,, ,, ,, | 53 | | 33 530
739 | ,, | | ,, | 240 | 7.52 | ,, 8″ to 10″ | 78 | | 33 390
750 | ,, | | ,, | ,, | 7.53 | ,, ,, ,, 10.1″ | 79 | | 34 390
1643 | ,, | | ,, | 320 | 7.69 | ,, 8″ to 10.75″ | 105 | | 30 840
1644 | ,, | | ,, | ,, | 7.70 | ,, 8″ ,, 10.85″ | 107 | | 30 770
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 9.



Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, In Feet	Area Sq. In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
379	Iron		Average 62 000 (Probably)	14.8	17.01		48		54 900
380	"			"	17.80		48		35 775
346	"			"	15.74		50		32 850
347	"			"	15.84		50		34 000
342	"			24.8	15.68		74		34 000
344	"			"	15.56		74		34 505
348	"			16.8	21.62		52		53 982
349	"			"	21.46		52		55 954
341	"			24.8	21.20		74		55 019
345	"			"	21.19		74		54 946
339	"			"	12.64		84		32 000
340	"			"	12.54		84		33 902
337	"			20.8	16.99		74		54 570
338	"			"	17.40		74		55 000
358	"			24.0	17.77		68		52 077
359	"			"	17.22		68		52 253
350	"			"	12.48		78		18 942
351	"			"	10.84		78		19 507
352	"			"	12.67		78		37 468
353	"			"	12.76		78		38 036

FIG. 10.



Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
377	Iron.		Average 50 000	128	12.08		48		31 722
378	"		(Probably)	"	11.11		48		33 564
367	"		"	180	11.42		65		32 329
368	"		"	"	11.42	"	65		33 205
356	"		"	240	11.42	"	87		29 947
357	"		"	"	11.31	"	87		29 186
371	"		"	120	9.11		50		31 302
372	"		"	"	10.07	"	50		31 698
370	"		"	180	9.21	"	74		31 650
369	"		"	"	9.44	"	74		30 720
354	"		"	240	9.24	"	99		28 950
355	"		"	"	9.36	"	99		29 879

FIG. 11.

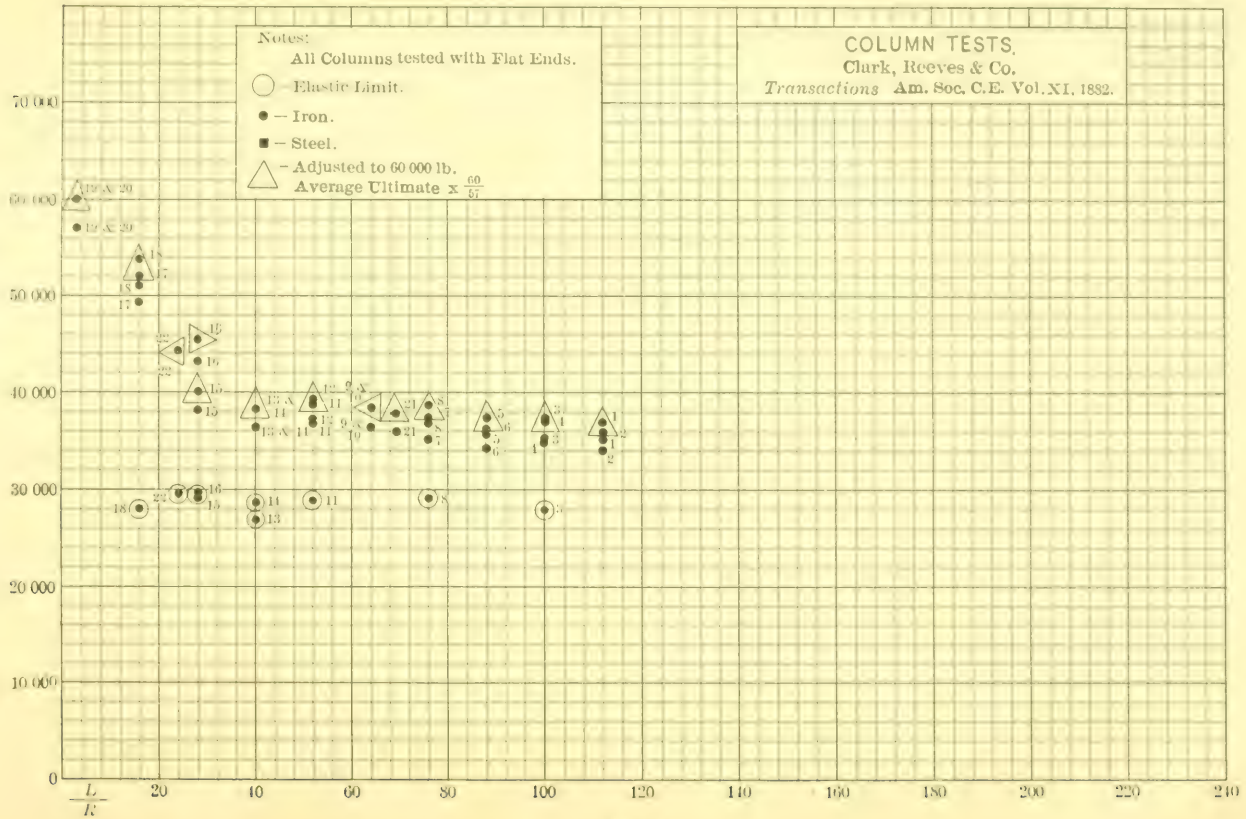
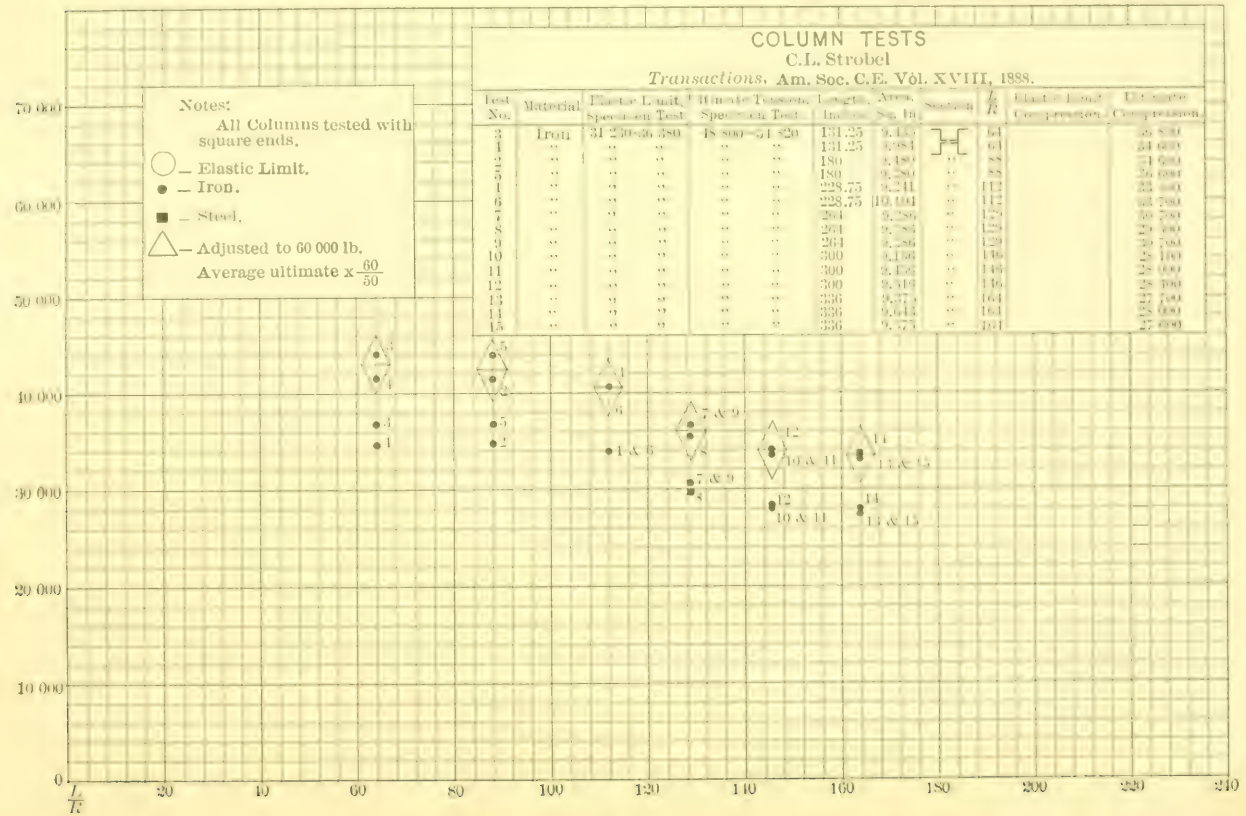

Notes:
All Columns tested with Flat Ends.
Elastic Limit.
– Iron.
– Steel.
– Adjusted to 60 000 lb.
Average Ultimate x 60/57
COLUMN TESTS.
Clark, Reeves & Co.
Transactions Am. Soc. C.E. Vol. XI, 1882.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 12.

COLUMN TESTS
Clark, Reeves & Co.
Transactions, Am. Soc. C.E. Vol. XI, 1882.

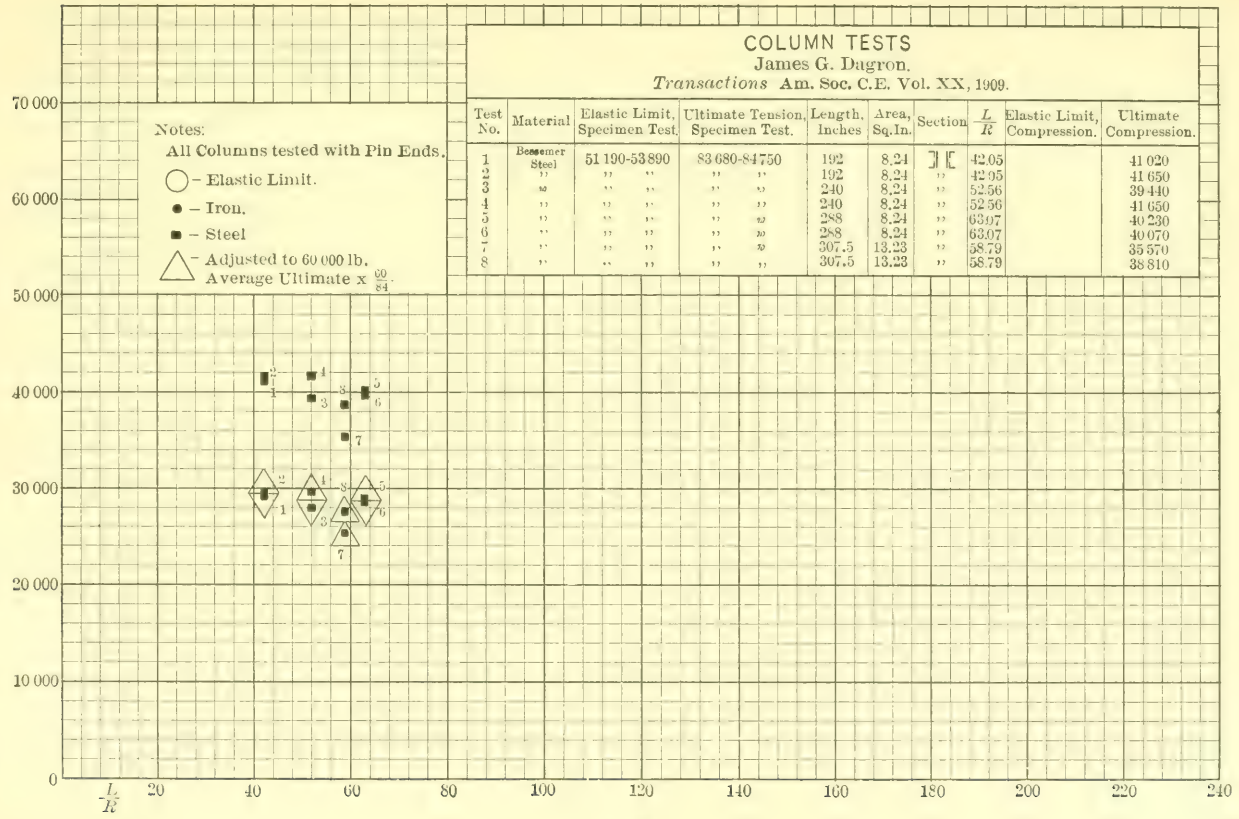
Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression.
1	Iron		Clark, Reeves & Co's Specification of 1871 Calls for 55 000 to 60 000	336	12.062	[illegible]	112		35 150
2	"			336	12.181	"	112		34 150
3	"			300	12.233	"	100	27 960	35 270
4	"			300	12.100	"	100		35 040
5	"			264	12.371	"	88		35 570
6	"			264	12.311	"	88		34 360
7	"			228	12.023	"	76		35 365
8	"			228	12.087	"	76	29 290	36 900
9	"			192	12.000	"	64		36 580
10	"			192	12.000	"	64		36 580
11	"			156	12.185	"	52	28 890	36 857
12	"			156	12.069	"	52		37 200
13	"			120	12.248	"	40	26 940	36 480
14	"			120	12.339	"	40	28 360	36 397
15	"			84	12.265	"	28	29 350	38 157
16	"			84	11.962	"	28	29 590	43 360
17	"			48	12.081	"	16		49 500
18	"			48	12.119	"	16	28 050	51 240
19	"			8	11.903	"	2.67		57 130
20	"			8	11.903	"	2.67		57 300
21	"			302.65	18.300	[illegible]	68.8		36 010
22	"			105.90	18.300	"	24	29 510	42 180

FIG. 13.



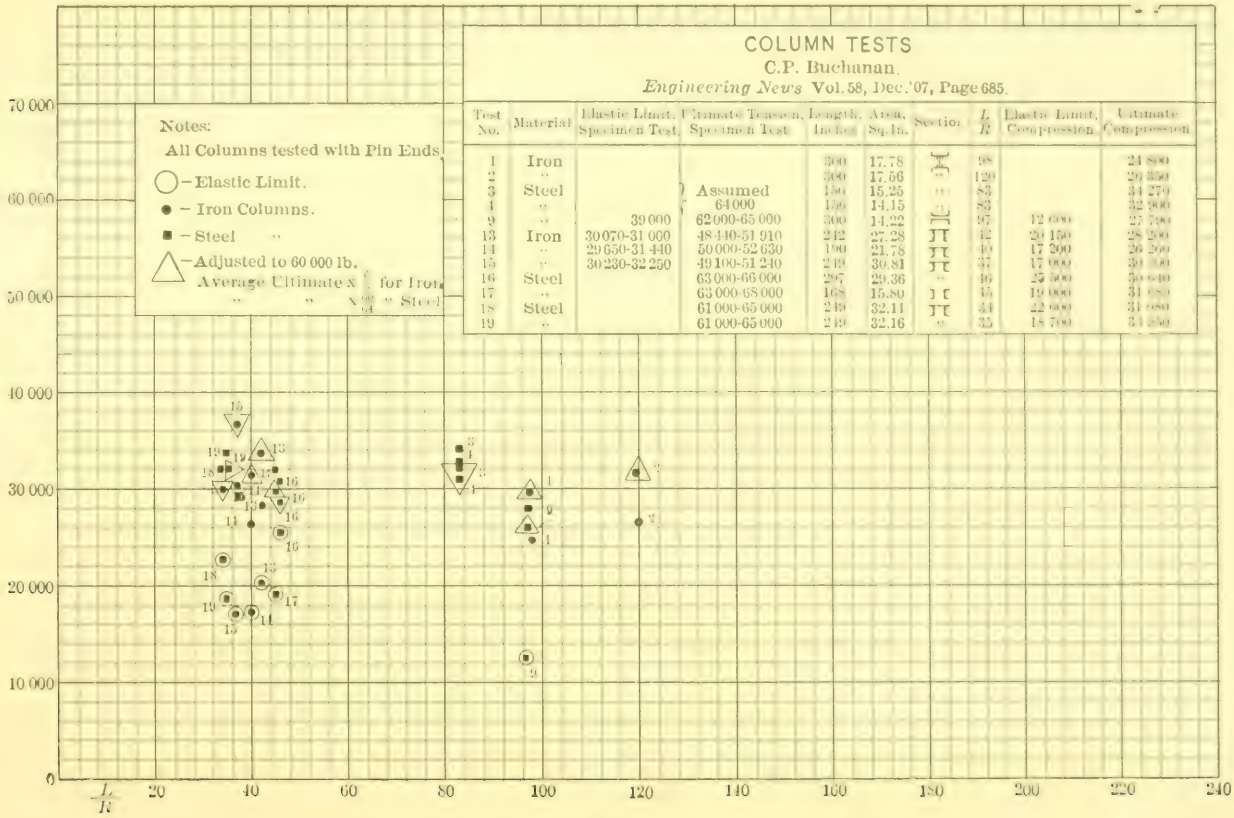
Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	l/r	Elastic Limit, Compression	Ultimate Compression
3	Iron	31 250–36 380	48 800–54 820	131.25	9.43	⊐⊏	64		36 820
1	"	"	"	131.25	9.984	"	64		54 620
5	"	"	"	180	9.489	"	88		41 620
2	"	"	"	180	9.280	"	88		26 080
4	"	"	"	228.75	9.241	"	112		22 440
6	"	"	"	228.75	10.004	"	112		34 780
7	"	"	"	264	9.286	"	129		30 780
8	"	"	"	264	9.286	"	129		32 160
9	"	"	"	264	9.286	"	129		30 780
10	"	"	"	300	9.156	"	146		25 160
11	"	"	"	300	9.156	"	146		25 160
12	"	"	"	300	9.319	"	146		28 300
13	"	"	"	336	9.377	"	164		27 780
14	"	"	"	336	9.643	"	163		28 060
15	"	"	"	336	9.573	"	164		27 000

Fig. 14



Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq.In.	Section	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1	Bessemer Steel	51 190-53 890	83 680-84 750	192	8.24	][	42.05		41 020
2	,,	,, ,,	,, ,,	192	8.24	,,	42.95		41 650
3	,,	,, ,,	,, ,,	240	8.24	,,	52.56		39 440
4	,,	,, ,,	,, ,,	240	8.24	,,	52.56		41 650
5	,,	,, ,,	,, ,,	288	8.24	,,	63.07		40 230
6	,,	,, ,,	,, ,,	288	8.24	,,	63.07		40 070
7	,,	,, ,,	,, ,,	307.5	13.23	,,	58.79		35 570
8	,,	,, ,,	,, ,,	307.5	13.23	,,	58.79		38 810

FIG. 15



Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
1	Iron			300	17.78	H	98		24 880
2	..			300	17.56	..	120		29 350
3	Steel		Assumed	150	15.25	..	83		34 270
4	..		64 000	150	14.15	..	83		32 900
9	..	39 000	62 000-65 000	300	14.22	H	97	12 690	25 780
13	Iron	30 070-31 000	48 440-51 910	242	27.28	][	42	20 150	28 200
14	..	29 650-31 440	50 000-52 630	190	21.78	][	40	17 200	26 200
15	..	30 230-32 250	49 100-51 240	249	30.81	][	37	17 000	30 [illegible]
16	Steel		63 000-66 000	297	29.36	..	46	25 500	30 [illegible]
17	..		63 000-68 000	168	15.80	][	45	19 000	31 [illegible]
18	Steel		61 000-65 000	249	32.11	][	44	22 000	31 [illegible]
19	..		61 000-65 000	249	32.16	..	35	18 700	34 [illegible]

Fig. 16.

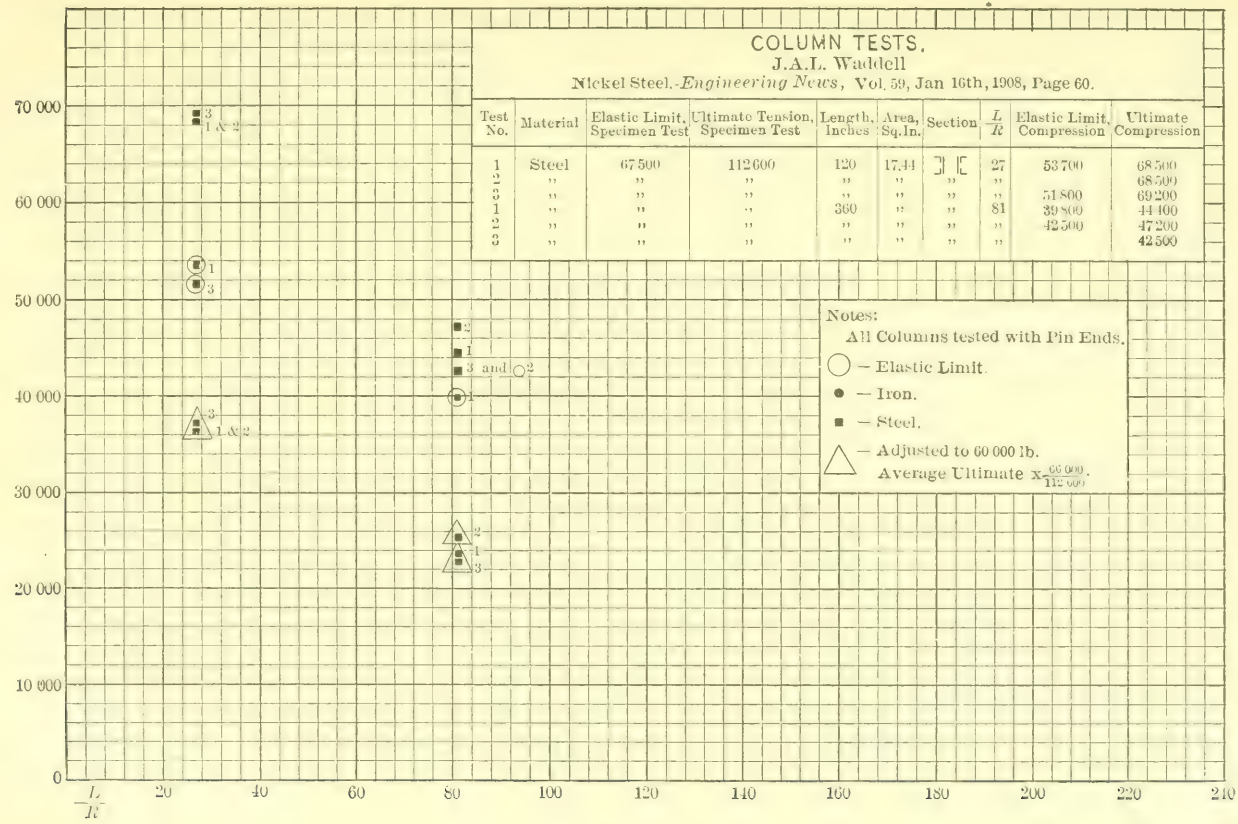

COLUMN TESTS.
J.A.L. Waddell
Nickel Steel.-Engineering News, Vol. 59, Jan 16th, 1908, Page 60.
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length, Inches | Area, Sq.In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression
1 | Steel | 67500 | 112600 | 120 | 17.44 |] [| 27 | 53700 | 68500
2 | ,, | ,, | ,, | ,, | ,, | ,, | ,, | | 68500
3 | ,, | ,, | ,, | ,, | ,, | ,, | ,, | 51800 | 69200
1 | ,, | ,, | ,, | 360 | ,, | ,, | 81 | 39800 | 44400
2 | ,, | ,, | ,, | ,, | ,, | ,, | ,, | 42500 | 47200
3 | ,, | ,, | ,, | ,, | ,, | ,, | ,, | | 42500
Notes:
All Columns tested with Pin Ends.
— Elastic Limit.
— Iron.
— Steel.
— Adjusted to 60 000 lb. Average Ultimate x 60 000/112 600.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240
3 1 & 2
1 3
2 1 3 and 1 2 1
3 1 & 2
2 1 3

FIG. 17.

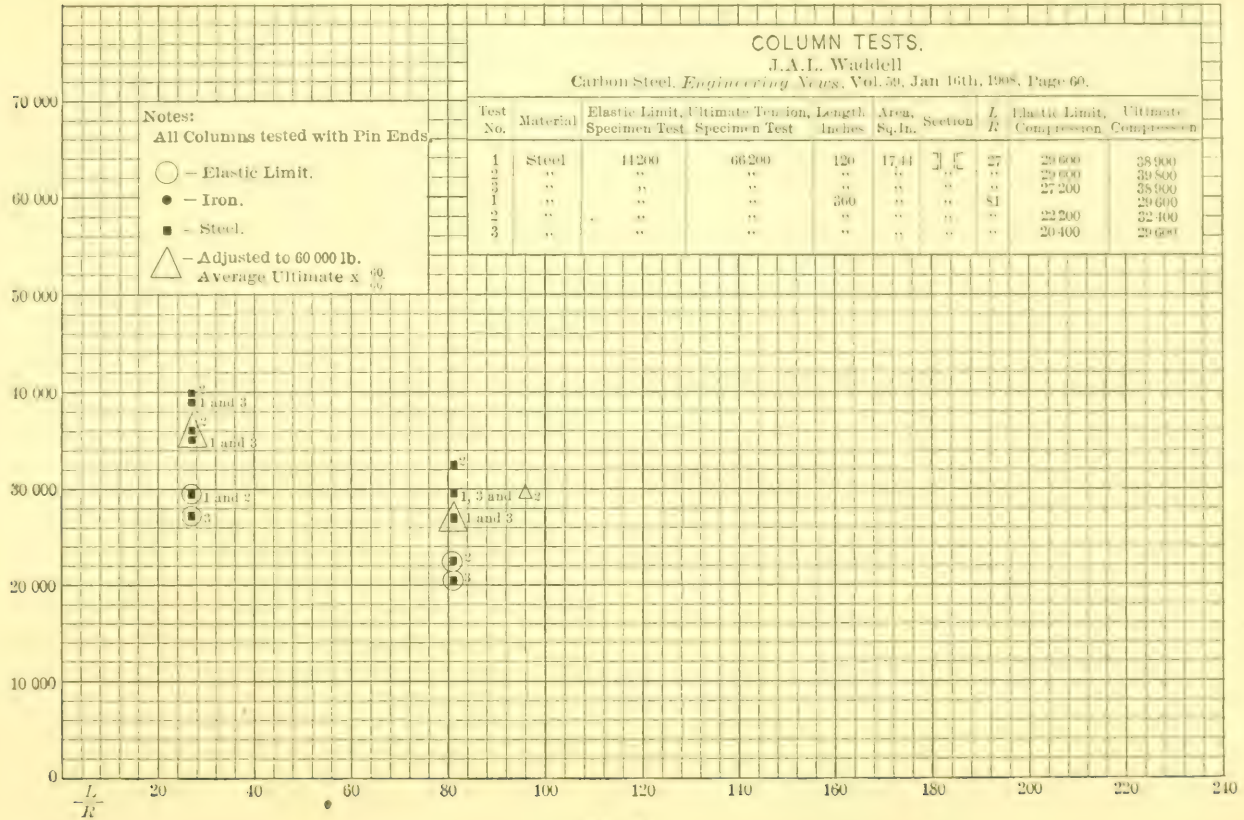

COLUMN TESTS.
J.A.L. Waddell
Carbon Steel. Engineering News, Vol. 59, Jan 16th, 1908, Page 60.
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length, Inches | Area, Sq. In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression
1 | Steel | 44200 | 66200 | 120 | 17.44 |] [| 27 | 29600 | 38900
2 | " | " | " | " | " | " | " | 29600 | 39800
3 | " | " | " | " | " | " | " | 27200 | 38800
1 | " | " | " | 360 | " | " | 81 | 29600 |
2 | " | " | " | " | " | " | " | 22200 | 32400
3 | " | " | " | " | " | " | " | 20400 | 29600
Notes:
All Columns tested with Pin Ends.
– Elastic Limit.
– Iron.
– Steel.
– Adjusted to 60 000 lb. Average Ultimate x 60/66
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240
1 and 3
1 and 3
1 and 2
3
1, 3 and 2
1 and 3
2
3

FIG. 18.

COLUMN TESTS.

Lap-Welded Steel Tubing.

Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension Specimen Test	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
1895	Steel	35 000-38 610	56 390-60 830	20.56	5.15	○	12.5	32 000	67 000
6	"	" "	" "	"	5.24	"	12.5	33 000	66 350
7	"	" "	" "	"	5.24	"	12.5	33 000	66 180
1891	"	" "	" "	41.00	5.34	"	25.0		41 700
2	"	" "	" "	"	5.25	"	"	30 000	40 610
3	"	" "	" "	"	5.17	"	"	29 000	39 850
1898	"	" "	" "	41.12	5.10	"	"	31 000	46 000
9	"	" "	" "	"	5.18	"	"	31 000	43 260
1900	"	" "	" "	"	5.36	"	"	29 000	41 420
1	"	" "	" "	61.69	5.06	"	37.5	32 000	34 000
2	"	" "	" "	"	5.23	"	"	30 000	32 000
3	"	" "	" "	"	5.34	"	"	27 000	31 740
1888	"	" "	" "	82.25	5.24	"	50		28 820
9	"	" "	" "	"	5.19	"	"		32 000
1890	"	" "	" "	"	5.21	"	"		32 000
1904	"	" "	" "	"	5.33	"	"	28 000	30 000
5	"	" "	" "	"	5.06	"	"	31 000	32 000
6	"	" "	" "	"	5.25	"	"	30 000	32 000
7	"	" "	" "	102.81	5.20	"	62.5	32 000	35 630
8	"	" "	" "	"	5.08	"	"		31 000
9	"	" "	" "	"	5.26	"	"	29 000	30 000
1894	"	" "	" "	120.50	5.16	"	73.3		33 000
1885	"	" "	" "	123.50	5.23	"	75		33 000
6	"	" "	" "	"	5.19	"	"		33 000
7	"	" "	" "	"	5.20	"	"		31 000
1882	"	" "	" "	164.50	5.26	"	100		31 000
3	"	" "	" "	"	5.21	"	"		28 000
4	"	" "	" "	"	5.17	"	"		31 000
1879	"	" "	" "	205.50	5.21	"	125		29 000
80	"	" "	" "	"	5.07	"	"		26 030
1	"	" "	" "	"	5.21	"	"		29 000
1876	"	" "	" "	246.75	5.11	"	150		26 000
7	"	" "	" "	"	5.17	"	"		27 000
8	"	" "	" "	"	5.16	"	"		23 000
1873	"	" "	" "	288.00	5.04	"	175		25 000
4	"	" "	" "	"	5.06	"	"		24 000
5	"	" "	" "	"	5.20	"	"		27 000

Average: 36 250

Average: 58 310

FIG. 19.

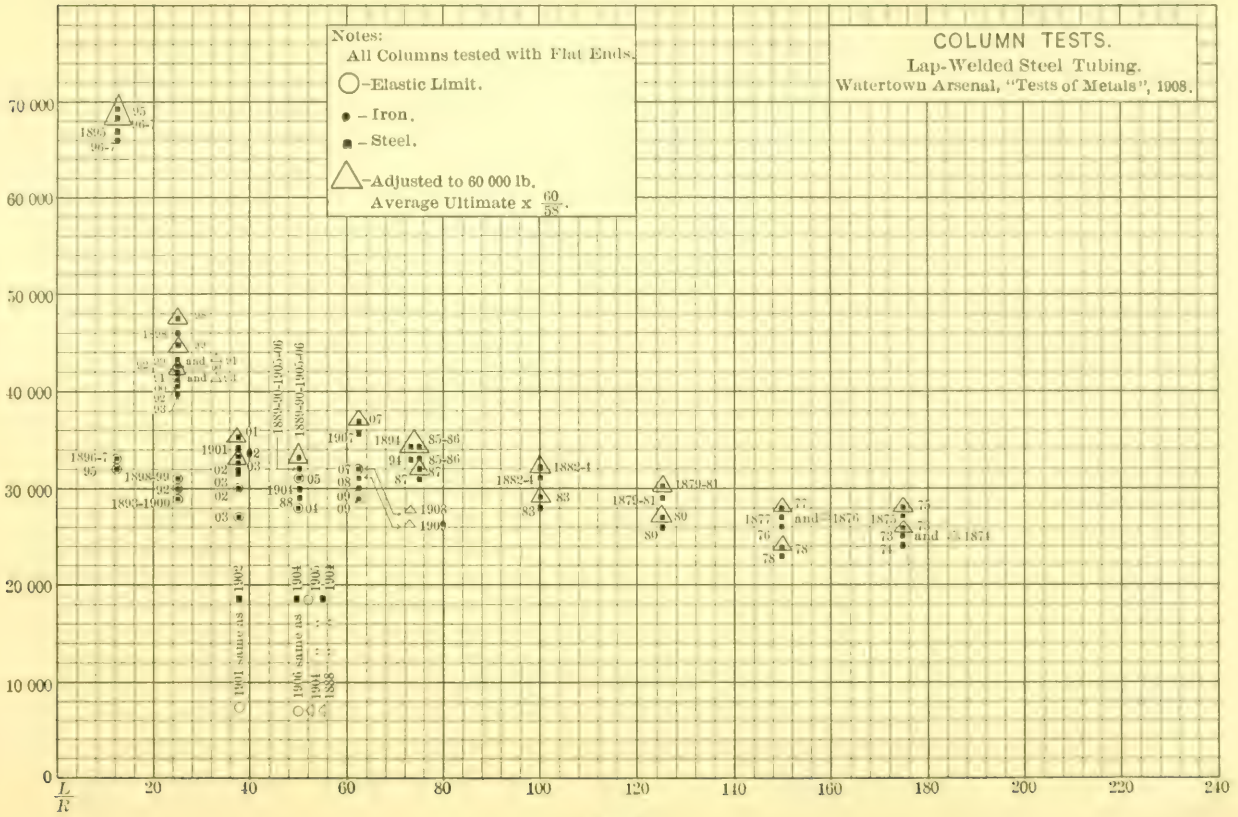
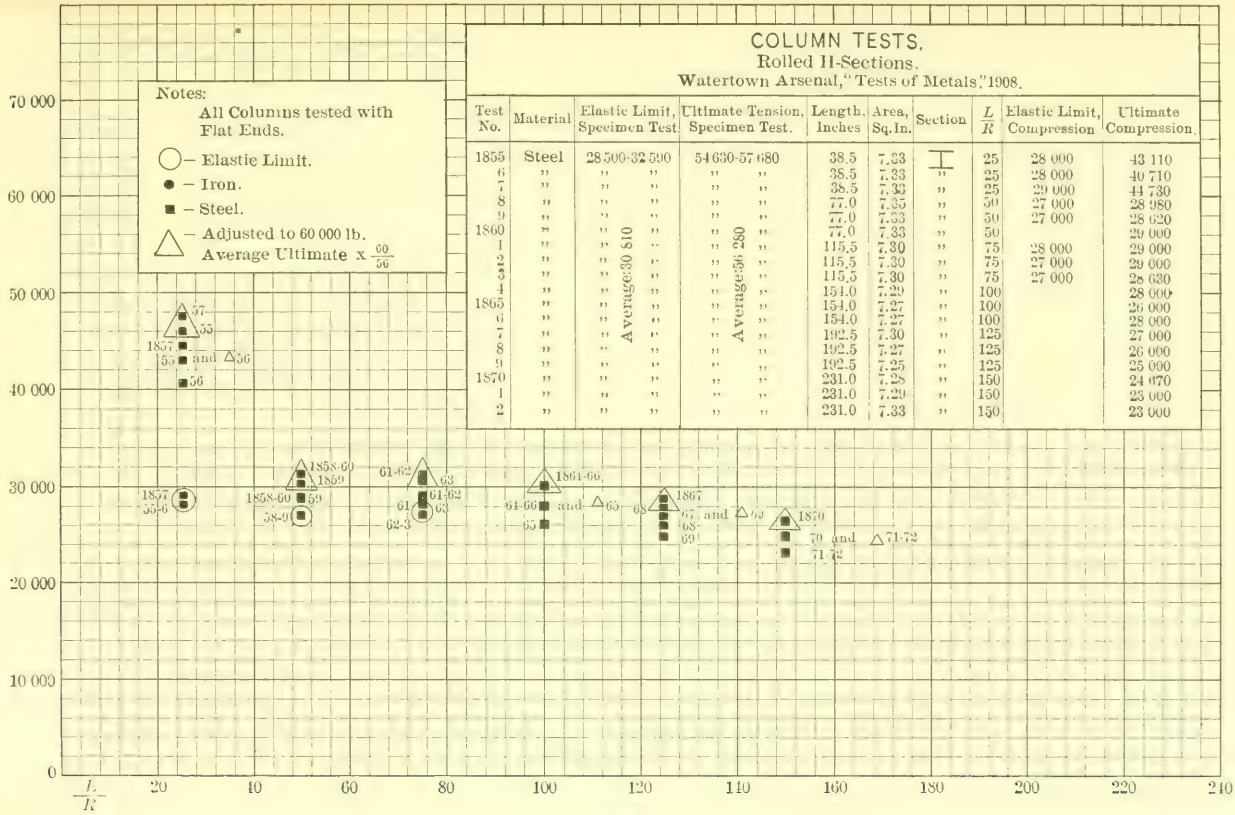

Notes:
All Columns tested with Flat Ends.
○–Elastic Limit.
• – Iron.
■ – Steel.
△–Adjusted to 60 000 lb.
Average Ultimate x 60/58.
COLUMN TESTS.
Lap-Welded Steel Tubing.
Watertown Arsenal, "Tests of Metals", 1908.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 20



COLUMN TESTS.
Rolled H-Sections.
Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression.
1855	Steel	28 500-32 590	54 630-57 680	38.5	7.33	I	25	28 000	43 110
6	,,	,, ,,	,, ,,	38.5	7.33	,,	25	28 000	40 710
7	,,	,, ,,	,, ,,	38.5	7.33	,,	25	29 000	44 730
8	,,	,, ,,	,, ,,	77.0	7.33	,,	50	27 000	28 980
9	,,	,, ,,	,, ,,	77.0	7.33	,,	50	27 000	28 620
1860	,,	,, ,,	,, ,,	77.0	7.33	,,	50		29 000
1	,,	,, ,,	,, ,,	115.5	7.30	,,	75	28 000	29 000
2	,,	,, ,,	,, ,,	115.5	7.30	,,	75	27 000	29 000
3	,,	,, ,,	,, ,,	115.5	7.30	,,	75	27 000	28 630
4	,,	,, ,,	,, ,,	154.0	7.29	,,	100		28 000
1865	,,	,, ,,	,, ,,	154.0	7.27	,,	100		26 000
6	,,	,, ,,	,, ,,	154.0	7.27	,,	100		28 000
7	,,	,, ,,	,, ,,	192.5	7.30	,,	125		27 000
8	,,	,, ,,	,, ,,	192.5	7.27	,,	125		26 000
9	,,	,, ,,	,, ,,	192.5	7.25	,,	125		25 000
1870	,,	,, ,,	,, ,,	231.0	7.29	,,	150		24 070
1	,,	,, ,,	,, ,,	231.0	7.29	,,	150		23 000
2	,,	,, ,,	,, ,,	231.0	7.33	,,	150		23 000

Average: 30 810 — Average: 56 260

FIG. 21.

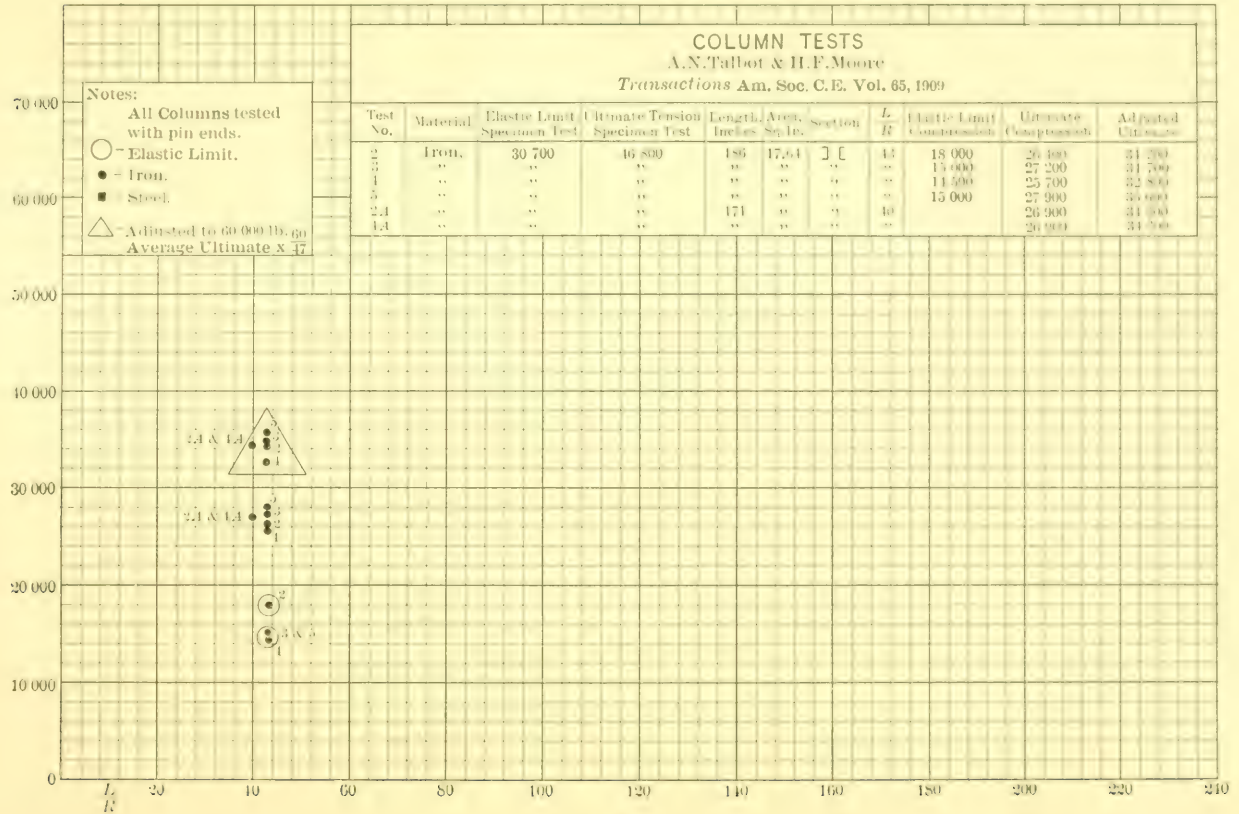

COLUMN TESTS
A.N.Talbot & H.F.Moore
Transactions Am. Soc. C.E. Vol. 65, 1909
Notes:
All Columns tested with pin ends.
Elastic Limit.
Iron.
Steel.
Adjusted to 60 000 lb. 60 / Average Ultimate x 47
Test No. | Material | Elastic Limit Specimen Test | Ultimate Tension Specimen Test | Length, Inches | Area, Sq. In. | Section | L/R | Elastic Limit Compression | Ultimate Compression | Adjusted Ultimate
2 | Iron. | 30 700 | 46 800 | 186 | 17.64 |] [| 43 | 18 000 | 26 400 | 31 200
3 | " | " | " | " | " | " | " | 15 000 | 27 200 | 31 700
4 | " | " | " | " | " | " | " | 14 500 | 25 700 | 32 800
5 | " | " | " | " | " | " | " | 15 000 | 27 900 | 35 600
2A | " | " | " | 171 | " | " | 40 | | 26 900 | 31 500
4A | " | " | " | " | " | " | " | | 26 900 | 34 700
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240
2A & 4A
2A & 4A
3 & 5
2
1

FIG. 22.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV.

JANUARY TO DECEMBER, 1909.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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UNDERPINNING THE CAMBRIDGE BUILDING, NEW YORK CITY.

By T. Kennard Thomson, M. Am. Soc. C. E.

To be Presented February 2d, 1910.

In July, 1907, the writer was retained by Henry G. Opdycke, Assoc. M. Am. Soc. C. E., to report on the undermining of the Cambridge Building, on the southwest corner of Thirty-third Street and Fifth Avenue, New York City, by the Pennsylvania Railroad Tunnel, then under construction.

Mr. Opdycke had already made a careful survey of the premises, and also had a number of excellent photographs, from which those illustrating this paper have been selected.

Fig. 1 shows the approximate cross-section of the tunnel excavation at a point opposite this building, as taken by Mr. Opdycke and the writer. The space marked "To be back-filled" represents the approximate deviation from the theoretical tunnel-lines, and shows that at one point an apparently misdirected blast caused a cavity extending some 3 ft. south of the building line of the Cambridge Building, or 9 ft. farther south than intended.

It also indicates roughly where Thirty-third Street caved in, on December 14th, 1906, in front of the Waldorf. The cave-in had been

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back-filled, but the curb, the street, and the Cambridge Building had continued to settle.

Fig. 1, Plate CXX, shows very plainly the settlement in Thirty-third Street. The street had been flooded, and the water covered the curb, in front of the entrance and near the middle of the Cambridge Building, which was the point where the greatest settlement occurred. The tendency of the whole building, as shown by all the photographs, was to bulge toward the north, as well as to settle vertically.

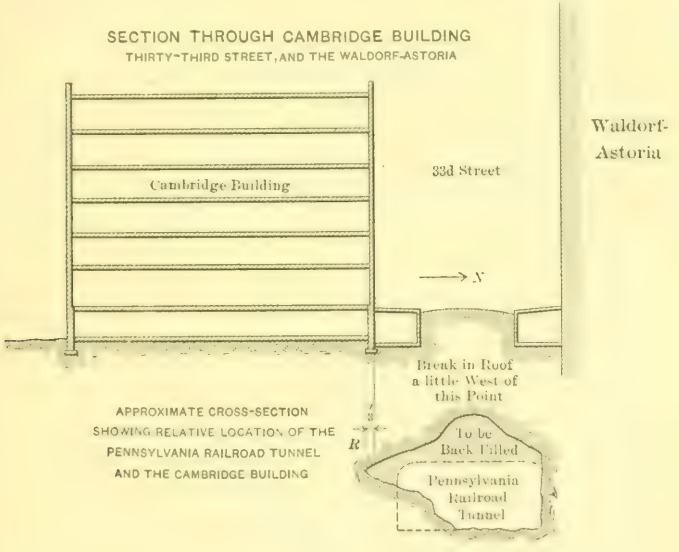


FIG. 1.

It was claimed by others that the whole trouble was caused by a defective 8-in. column just east of the entrance, but it is peculiar that, although this column had moved northward and had settled, it did not break its bed nor its connection with the brickwork of the north wall, and remained absolutely plumb.

There were a number of vertical and inclined cracks in the interior north and south walls, showing the tendency of the building to move northward and at the same time to settle vertically, the dimensions of the cracks decreasing as the distance from the curb increased; in fact, the greatest disturbance in the building itself was at the center of the north wall, and in the areaway, sidewalk, and street, as might be expected.

The connection between the two boilers in the cellar parted, break-

ing the pipe connections. Both boilers settled, the one several inches farther north than the other.

There were several cracks in the area floor, and some disturbance of the area wall, one joint alone opening up more than 1 in.

The sidewalk settled considerably, as shown in Fig. 2, Plate CXX; and there were some cracks. The curb was reset several times, sometimes as much as 7 or 8 in.

Fig. 1, Plate CXXI, shows some of the cracks in Kirkpatrick's Jewelry Store, in the Cambridge Building. In a number of places the joints in the marble work in the main hall opened up.

The fact that an old stream ran under the site of the Cambridge Building and the Waldorf-Astoria, also aggravated matters, especially as the break in the tunnel was at or near this old stream. The foundations of the Waldorf-Astoria had been carried down to the rock, but the east end of the Cambridge Building was founded on a spread footing of concrete and the west end on piles.

When the Cambridge Building was remodeled, in 1906, to convert it from a hotel into an office building, these piles, which were about 30 ft. long and, when driven, had been cut off below the water line, were found to be slightly above the water line, the latter having been lowered enough to expose their tops to the atmosphere, allowing them to rot.

It is known, of course, that timber will last practically forever if entirely submerged, but will soon rot out if not entirely under water. Therefore, at that time, the piles were cut off below the new water line, capped with concrete, and five heavy 20-in. I-beams, about 40 ft. long, were placed on the concrete and buried in it, making a very substantial foundation for the west end of the building.

In 1907, when Mr. Opdycke and the writer examined the tunnel, they noticed that several rather large streams of water—evidently from the old stream shown on the maps—were pouring into it at this point. The examinations showed that the building had settled considerably and was still settling, and the writer was requested to design a system of underpinning carried to rock. This he refused to do at that time, for fear of opening up a direct connection between the cylinders and the tunnel, thus endangering the entire building; but he recommended tying back the front wall with rods, to prevent it from falling away from the remainder of the building.

FIG. 1.—SETTLEMENT OF STREET, CURB, ETC., IN THIRTY-THIRD STREET, IN FRONT OF CAMBRIDGE BUILDING.

FIG. 2.—SETTLEMENT OF SIDEWALK, IN THIRTY-THIRD STREET, IN FRONT OF CAMBRIDGE BUILDING.

This was immediately done, as shown by Fig. 2. Two 12-in. beams, about 41 ft. long, were placed vertically just east of the main entrance on Thirty-third Street, which was the point of greatest disturbance. These beams extended from the areaway floor up to and slightly above the third floor. The tops of the beams were anchored back by a 1½-in. rod, about 102 ft. long, made up of stock lengths of about 16 ft., and connected with turnbuckles.

The rear or south end of the rods ran through an open court, but close to the wall. On each side of the court 12-in. I-beams, about 6 ft.

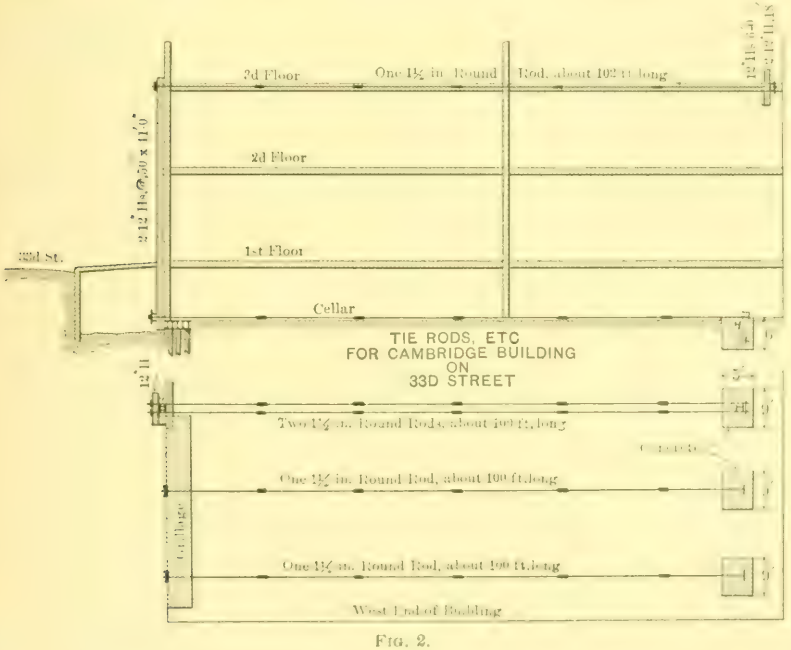

3d Floor
One 1½ in. Round Rod, about 102 ft. long
2d Floor
1st Floor
Cellar
33d St.
TIE RODS, ETC
FOR CAMBRIDGE BUILDING
ON
33D STREET
Two 1¼ in. Round Rods, about 100 ft. long
One 1¼ in. Round Rod, about 100 ft. long
One 1¼ in. Round Rod, about 100 ft. long
West End of Building

FIG. 2.

long, were set vertically, and formed seats for the two 12-in. beams (18 ft. long) to which the rod was anchored. The north end of the rod was placed just under the floor in a private office, and through the main hall.

The bottom ends of these long beams were anchored with two 1¼-in. rods, just above the cellar floor. These rods were about 100 ft. long, and were made up of short sections, the rear or south ends being anchored to 5 by 9-ft. blocks of concrete, about 6 ft. deep, placed just under an old north and south partition wall, and in a position slightly

inclined from the vertical. Two horizontal beams were also placed in the concrete, as shown.

Near the west end of the building, another 1½-in. rod was placed, just above the cellar floor, the north end being hooked over the top of the five 20-in. I-beams, and the south end being anchored to a similar block of concrete.

Half way between this westerly rod and the first pair, a similar rod was placed and anchored in the same manner as the westerly rod.

The 41-ft. beams in front were then encased in sheet metal on account of their unsightly appearance.

The writer's examination was completed about July 23d. A contract was at once made with Mr. Alexander Brown, Jr., to tie back the building, and the work was completed in about a week.

The rods were left in place until the permanent underpinning had been completed (during the following year), and effectually prevented the north wall from pulling away, although the entire building had apparently moved north in the interval. The anchorages in the cellar, although made very heavy, showed signs of the great strain, by the cracks which opened up in the concrete floor behind them.

When the rods were first put in, in the autumn of 1907, the writer tested them to see that they were under sufficient initial strain, and when he was called in to make another report and examination, in the following spring, a similar test indicated that they were under a considerably greater strain, showing the tendency of the building to move north. In the fall of 1908, when these rods were to be removed, the men commenced to saw through the second rod from the west on the cellar floor, quite near the end attached to the 20-in. I-beams, without first slacking up the turnbuckles. When they had sawn three-quarters through the rod, it broke with a snap, and the long end sprang back about 2 ft. The men followed their instructions with the remainder of the rods, and slackened off the turnbuckles first.

The examination of the building, in March, 1908, indicated that there had been considerable vertical movement or settlement since the previous fall. Borings then taken showed that the rock was about 20 ft. below the curb at the east end of the building and about 44 ft. at the west end. The substrata were found to be partly of made ground, and below that there was sand, gravel, clay, and silt or muck, such as found in the beds of all old New York streams—a very treacherous material to have under a building.

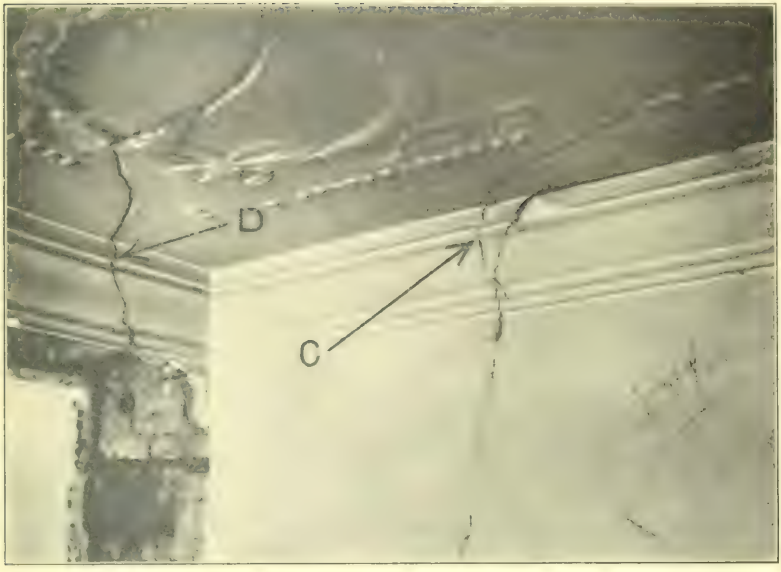


FIG. 1.—CRACKS IN WALL AND CEILING, MAIN FLOOR, CAMBRIDGE BUILDING.

FIG. 2.—SETTLEMENT IN AREA, CAMBRIDGE BUILDING.

The borings also indicated that the old stream had been entirely diverted by the tunnel, leaving the piles dry, and making their ultimate destruction by rot certain.

In the meantime the tunnel excavation had been completed, and the permanent masonry lining built in place, and it was thought that the stream which had been flowing into the tunnel the year before would back up, when it could not get into the tunnel, and again submerge the piles. The borings, however, indicated that this was not so, but that the tunnel had in some manner effectually and permanently diverted the old stream, thus lowering the ground-water level 30 ft. or more.

Reports had been obtained from H. G. Opdycke, Assoc. M. Am. Soc. C. E., Rudolph P. Miller, M. Am. Soc. C. E., Daniel E. Moran, M. Am. Soc. C. E., and Mr. Joseph Caccavajo, and, as these were similar in nature to that made by the writer, the Gorham Company (the lessee), decided to underpin the building to rock, and a contract for doing so was made with Mr. Alexander Brown, Jr., who retained the writer as Consulting Engineer to devise a plan and carry it out.

The east end of the north wall of the Cambridge Building was occupied by the show windows of the Kirkpatrick Jewelry Store, on the Fifth Avenue and Thirty-third Street sides of which store the old brick wall in the first story had been removed and a number of 8-in. cast-iron columns had been inserted to carry the upper brick wall and floors.

This is a kind of construction very frequently resorted to in converting old buildings into stores, etc., and answers the purpose as long as the foundations are not disturbed, or the walls jacked against, as there is no transverse bracing between the columns; therefore it was decided not to use the old system, which involved jacking against the building to force cylinders or walls into the ground.

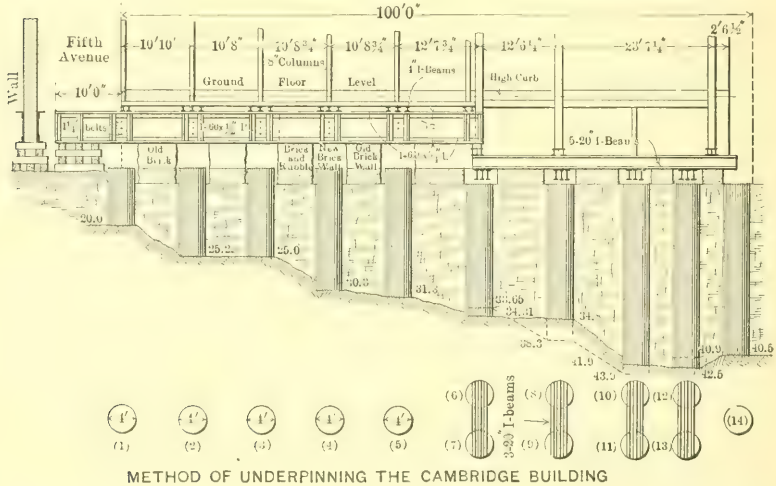
Under this end of the wall, from the curb to the cellar, there was a wall of brick and rubble masonry with some ashlar facing, which had been built, as was customary at the time and for that class of work, without any engineering supervision or advice; in other words, it was of the usual poor construction.

This foundation wall rested on a concrete footing, and on top of the wall were set the 4-in. beams, which for the length of the wall, formed a base for the 8-in. cast-iron columns.

As it was not advisable to do much cutting in this wall in the condition it was then in, it was decided to reinforce it by placing a 5-ft.

plate girder on each side about 1 ft. below the 4-in. beams, to bolt these two girders together with $1\frac{1}{4}$-in. bolts, and to grout the spaces between the girders and the wall. These spaces were usually 1 in. or more in width, as no attempt was made to dress down the surface of the masonry. As these girders in place were to be about 67 ft. long, they were each shipped in two sections in order that they might be put in position readily.

The west end, shown by Fig. 3, rested on the end of the old 20-in. steel beams, and the south end on a timber blocking under the sidewalk of Fifth Avenue.



METHOD OF UNDERPINNING THE CAMBRIDGE BUILDING

FIG. 3.

These girders had $\frac{1}{2}$-in. web plates and one 6 by 6 by $\frac{3}{4}$-in. angle in each flange.

After the underpinning was completed, the 10 ft. under Fifth Avenue was sawn off, and a space of about 4 ft. was cut out for a door just west of the splice, the splice having been placed at that point on account of the door.

The girders were placed a little more than 2 ft. 2 in. apart, back to back, and, with the exception of the south end and the space cut out for the door, were eventually bricked up on both sides, making a solid wall on top of the cylinders. As soon as the cylinders were in place and grouted, three 9-in. beams were placed under the 4-in. beams and under each column. The ends of these 9-in. beams were wedged up on

top of the 5-ft. girders, thus transferring the load from the columns directly to these girders. The holes around these short beams were then filled with concrete and brickwork. There were six 8-in. cast-iron columns resting on this pair of girders, and it was now considered safe to undermine two columns at a time in order to make room for the underpinning cylinders, one of which was put directly under each of the five most easterly columns.

As the sixth column and the remainder of the wall, or the portion from the east side of the entrance to the west end of the building, was being carried on the five 20-in. grillage beams, resting on the concrete-capped piles, it was thought that it would be too expensive and dangerous to try to put the cylinders directly under the columns, especially as it would have been necessary to cut out the old piles all the way down. Therefore it was decided to sink four pairs of cylinders, as close to the grillage beams as possible, and then remove enough concrete to place 20-in. I-beams under the old grillage beams, the ends of these short beams to rest on the pair of cylinders, and wedges to be driven between the short beams and old long grillage beams. Three of these short beams were inserted over each pair of cylinders. At the extreme end an additional cylinder was sunk on the center line of the wall, making, in all, fourteen cylinders for the support of the building.

Two additional cylinders were sunk in the areaway, to support the north end of the boilers and the area wall.

Mr. Brown called for bids on this plan, and awarded the contract to Thomas B. Bryson, Assoc. M. Am. Soc. C. E., as his bid was less than half as great as the other bids, and it was concluded that he could do a good job and make a fair profit, both of which he did. He also gave a bond to the owners for twice the amount of his contract. His total contract, including the extra work, was about $13 000, which included supplying and sinking the sixteen cylinders, filling them with concrete, supplying and placing the girders and beams, doing the necessary wedging, etc., and back-filling with concrete where necessary.

The cylinders (Fig. 4) were 4 ft. in diameter, and had very thin shells. The method of placing them constitutes a new system of underpinning; for, instead of placing the bottom section first and jacking it into the ground, and then placing another section on top and repeating the operation, a space was excavated for the top section, which was made 4 ft. deep, and the space outside of this top section was back-

filled, generally with concrete, and remained permanently in that position.

A laborer then entered the top section and, with a short-handled shovel, excavated the material under the "cutting edge" sufficiently to insert one segment of the next ring. All the rings below the top

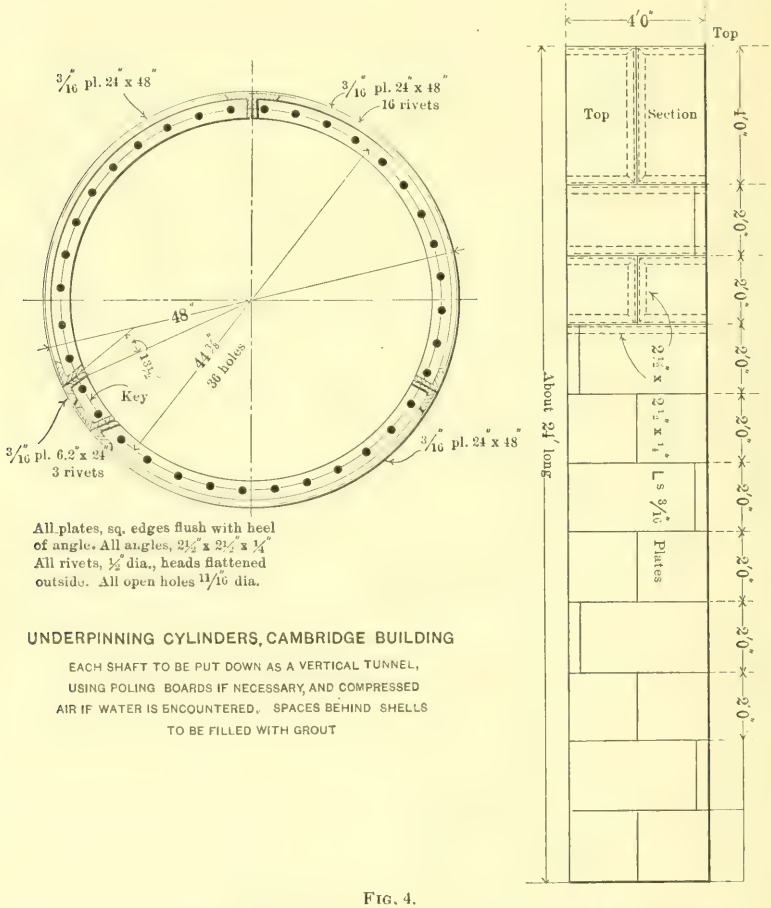


FIG. 4.

section were 2 ft. deep and in four segments, one of which was small, to act as a key, as in tunnel lining. In this key segment the connection angles were bent $13\frac{1}{2}^\circ$ from the radial, to permit putting the key in place. The joints of the other segments, of course, were on the radial lines. All the plates were $\frac{3}{16}$ in. thick, and all the angles (horizontal

and vertical) were 2½ by 2½ by ¼ in. When the first segment under the top section had been bolted in place, the excavation was made for the second segment, then the third, and then the fourth or key. Where the ground was good, the excavation was made for all four segments at once.

Although the borings showed that all the water had been drained off, it was considered desirable to provide in the contract for doing the work under compressed air if water should be encountered, and the first cylinder to be sunk had gaskets in the joints; but, as it was found that the borings had been correct in showing the absence of water, no effort was made to obtain water-tight joints in the other fifteen cylinders.

In sinking one of the cylinders the rock gave evidence of having been blasted, as it undoubtedly had been, as this cylinder was directly over the tunnel. The flushing of the street near the Waldorf allowed considerable water to percolate and get into the uncaulked joints of the cylinder, sometimes in quite a stream, but it was noted with surprise that this water disappeared as soon as it reached the bottom of the cylinder, passing into the drains or crevices alongside of the completed tunnel, although these drains were supposed to have been blocked off or choked up.

This proved the existence of the second danger feared for the Cambridge Building, resulting from the construction of the tunnel. The first was the rotting of the piles due to the removal of the groundwater, and the second the removal of the subsoil, especially where it was muck or silt, as in this case, by the continual draining of the water from the streets, old streams, etc., into or alongside of the tunnels, for it is known, of course, that such continual drainage must carry a large amount of soil or muck with it, causing a gradually enlarged cavity which, sooner or later, would be filled from above, perhaps suddenly, and with disastrous results.

This action is occurring in so many places in New York City that some day we may expect to see a whole building suddenly collapse, like some of the villages in mining districts.

These plans were drawn up in July, 1908; Mr. Bryson was awarded the contract early in August, and by August 21st he had the first cylinder on rock, having spent 4 or 5 days on it, as his men were new to the work, and extra precautions were taken against water, etc. The other

cylinders were put down at the rate of about 2 days each for a depth of about 30 ft. below the curb, the sixteenth or last cylinder being completed exactly 30 days after the first was landed. Generally, an attempt was made to work on two cylinders at a time, but in such cramped spaces it was difficult and subject to delays. Mr. Bryson sublet the placing of the four 33½-ft. girders to Mr. Frank M. Tench, who handled the ticklish problem without disturbing the sidewalk in front of the jewelry store. As the cellar under this store was a salesroom for valuable goods, such as statues, paintings, etc., it was necessary to build a tight wooden partition, about 5 ft. south of the wall which was reinforced by the girders, before work was started, and, after the girders had been bricked in, furred and plastered, the rich red burlap was replaced before the temporary partition was removed.

In order to get permission from the tenants, Mr. Brown had agreed to restore the premises of the Kirkpatrick Company by the end of September, on account of the fall trade, and, by working day and night, he succeeded in doing this.

Table 1 shows the order in which the cylinders were sunk, their depths, etc.

TABLE 1.

Order of sinking.	No. on plan.	Date of reaching rock.	Depth of rock below curb, in feet.
1	11	Aug. 21-08	41.9
2	10	" 26-08	43.9
3	8	" 29-08	38.3
4	9	" 31-08	34.8
5	13	Sept. 5-08	42.5
6	12	" 7-08	40.9
7	4	" 8-08	30.3
8	2	" 9-08	25.2
9	5	" 11-08	31.3
10	3	" 12-08	25.01
11	1	" 14-08	20.0
12	7	" 14-08	34.3
13	6	" 16-08	33.6
14	14	" 17-08	40.5
15	15	" 18-08	41.4
16	16	" 21-08	36.0

It might be stated here that although there was continual settlement up to the time that this building was underpinned, there has been absolutely none since. At one time, when the building was being tied back, the settlement of the area wall broke a 4-in. water main, which caused a good deal of worry, as the water started to flow around the

boilers, and soon would have found its way into the tunnel. It took 3 hours for the proper authorities to cut off this pipe. However, as soon as the pipe broke, the men made a dam of sand bags, thus keeping most of the water away from the low spot around the boilers and allowing it to escape into the sewer.

The writer wishes to thank those connected with this underpinning work, namely, Mr. John S. Holbrook, Vice-President of the Gorham Company; Henry G. Opdycke, Assoc. M. Am. Soc. C. E., Consulting Engineer; Mr. Alex. Brown, Jr., General Contractor; Mr. Frank M. Tench, Erecting Contractor; the Cockburn Barrow and Machine Company, which supplied the steel cylinders, and Lewis F. Shoemaker and Company, which supplied the steel girders, for the way in which each did his share in stopping the settlement of this building.

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BUILDING AGREEMENTS.

By William B. Bamford, Assoc. M. Am. Soc. C. E.

To be Presented February 2d, 1910.

This paper has been limited to agreements for the construction of buildings, and no attempt has been made to cover the subject of agreements in general, because the writer has given this type special study and also feels that the principles underlying such agreements are readily adaptable to those for other construction, and that, within the limits of this paper, many of the vital questions at issue can be brought out more effectively by taking a concrete case rather than by dealing in generalities.

From the practical standpoint of the building agreement, it matters little whether the person who planned the building and will have charge of it until completion is called an Engineer or an Architect. The only practical difference would be that the word "Engineer" would be used in one case and the word "Architect" in the other, to represent such a person. Therefore, it should be understood that wherever the word "Engineer" is used throughout this paper it is intended to designate a person properly equipped by education, experience, training, or temperament, to lay out, supervise, and direct, in an executive capacity, all work, at any stage, in the erection of a building, from the time it is mentally conceived until it is finished and ready for use.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The purpose of this paper is not a defense of the Contractor, as against the Engineer or Architect, but is simply one advocating justice and equity to all parties to a contract. Destructive criticism is easy, but constructive criticism is of a different mould, and should always be welcomed among broad-minded men. In suggesting a new form of Agreement and Schedule of Conditions of Contract, it is hoped that something definite may be accomplished in righting the wrongs that most Engineers will readily admit exist, and nearly everyone has tried by different means to correct. The writer does not imagine that he has discovered some new and patentable truth, but hopes that his studies may have brought into concrete shape some practical suggestions for correcting well-recognized faults. After Engineers and Architects have set about putting their own houses in order, as typified by contract requirements, they can then more fearlessly demand that Contractors right the wrongs for which they alone are to blame.

Much might be said on the general subject of fairness and justice to both Contractor and Owner in the various contract requirements, on the faults of omission and commission by Engineers and Architects, and on their duties, obligations, and responsibilities. These subjects would have a direct bearing on the principles underlying the reasons for many of the contract requirements herein suggested. The legal questions involved in contract requirements are also of great importance. In order, however, to keep this paper within reasonable bounds, its subjects have been abridged, but reference has been made to articles and books where much additional information may be obtained, or where some subjects, only touched upon here, will be found in more amplified form.

Historical Summary.

In gathering material for a prospective book on Building Specifications and Contracts, numerous recent specifications and contracts were specially obtained from many prominent architects and engineers for the purpose of study and examination. A careful examination of these, as actual working instruments, and a study of recently published works or addresses, have led to the belief that there was probably a need for investigating and discussing the subject of Agreements (or so-called Contracts) in general, and Building Agreements in particular.

The need for a more or less authoritative set of requirements,

applicable to a proper building agreement, has been recognized for many years. To meet such need, the American Institute of Architects, in co-operation with the National Association of Builders, adopted, a number of years ago, the "Uniform Contract." This contract has remained in force, with no very material changes, up to the present time. The last revision was copyrighted in 1905. This contract, with all its defects, is in general use among Architects, as there is no other form, having the endorsement of a representative American organization, to take its place.

In justice to the American Institute of Architects, it should be said that a Standing Committee on the Standardization of Documents was appointed in February, 1906, and has done much excellent work in collecting material for a new building agreement. Early in 1908 the Committee published, for limited circulation, a pamphlet containing a tentative series of forms, to facilitate the making of suggestions or criticisms to assist the Committee in its further study.

It was recently learned from several authoritative sources that the work of the Committee might not be published for some time, possibly years, and, as the writer's work has been entirely independent of the Committee, he has decided to present its results to this Society for discussion.

Several years ago, the writer learned that the Royal Institute of British Architects had spent many years in perfecting a just and equitable building agreement, but investigation revealed the fact that although the Institute had commenced the work more than thirty years ago, it was almost unknown in America.

The more closely the writer analyzed his own work and ideas and compared them with those of the R. I. B. A., the more firmly convinced he became of the excellence of the latter's work. Looking back at the time which has been spent in perfecting this work, and the number of eminent and experienced men who have contributed to it, there should be small wonder at its present superior condition.

While an agreement for which originality could be claimed for the whole, might be of greater personal benefit, it was decided to sink such personal consideration and frankly undertake to revise a most excellent document, to meet American conditions. The Agreement and the Schedule of Conditions of Contract put forward by the writer have been modified, after serious study, from those adopted by the

R. I. B. A., and the language of the original has been retained wherever possible. Such a revised document has a very important practical advantage over an attempt at originality. As many of the requirements vitally modify traditional American practice in many respects, it might be objected to because it has not had the test of sufficient experience to make it advisable for general use. English and American jurisprudence are fundamentally the same, and therefore the laws or judicial decisions of one country are more nearly similar to those of the other than is true of any other two countries. Since the work of the R. I. B. A. was undertaken originally about 1878, and the present documents were approved in substantially their present shape in 1895, and approved in their present shape in 1903 by the Institute of Builders and the National Federation of Building Trades Employers of Great Britain and Ireland, as well as by the Royal Institute of British Architects, they have had, in Great Britain, the test of time and of litigation, and can therefore be accepted with greater confidence as a secure foundation upon which to erect the desirable superstructure.

The following historical sketch of the British document, as given in an address by the President of the Institute, Mr. Aston Webb, on June 13th, 1903, is of much interest:

"The first time the subject was broached was on the 18th January, 1887, when Mr. T. Rickman announced at a Meeting of the Practice Committee that he had been approached by the Secretary of the Institute of Builders with a view to alterations in the existing Heads of Conditions of Contract. The Committee agreed to consider the question, and a Sub-Committee was formed. The Sub-Committee submitted their draft of General Conditions to the Practice Committee on the 18th December 1888, and for the following year the draft was considered by the Practice Committee. Negotiations were then instituted (December 1889) with the Institute of Builders. On the 29th May 1891 the Committee considered the draft as altered by the Institute of Builders, and negotiations with that Body were continued through the year. On the 9th August 1892 the first Conference between the Practice Committee and the Institute of Builders was held, the representatives of the Practice Committee being Messrs. Henry Currey, H. C. Boyes, G. E. Grayson, Edwin T. Hall, and T. M. Rickman. Negotiations then continued for two or three years, but unfortunately it was found impossible to come to an agreement. In 1895 the negotiations were dropped, and in July of that year the Institute Form of Contract, which was in use from that date to April, 1901, was issued.

"On the 5th December 1898 the Secretary of the Institute reported that he had had an interview with the Secretary of the Institute of Builders with reference to an agreement between the two Bodies on the Arbitration Clause, and a Sub-Committee of the Council, consisting of Messrs. Aston Webb, E. A. Gruning, John Slater, Wm. Emerson, Thomas Blashill and Edwin T. Hall, was appointed to confer with the Builders. Several meetings of the joint representatives were held, and on the 8th October 1900 the Institute of Builders forwarded the Report of the Joint Committee to the Council and stated that they were prepared to adopt it. On the 10th December the Council submitted the revised Paper to the General Body. The Arbitration Clause, however, as revised, met with such strong opposition that the Council referred the matter again to the Institute of Builders urging the modifications required to make the Clause acceptable to the R. I. B. A. After some further correspondence it was announced in April 1901 at a General Meeting that the negotiations between the Council and the Institute of Builders for a uniform Contract had failed, and at a Special General Meeting held on the 15th April 1901 the present Form as revised by the R. I. B. A. was sanctioned. On the 19th January 1903 the President reported to the Council that the President of the Institute of Builders had expressed the willingness of the Builders to accept the Institute Form as it now stood, with the addition of a proviso to Clause 16. The matter was referred to a Committee of the Council, who on the 18th May 1903 reported that they had conferred with Mr. King and Mr. Greenwood on behalf of the Builders, who had agreed to accept the Institute Form subject only to the modifications now proposed. The President of the Institute of Builders was then requested to send a formal communication to the Council stating that his Institute and the National Federation of Master Builders were prepared to accept the two Forms of Agreement and Conditions issued by the R. I. B. A. provided the alterations were agreed to; and that the Council would then recommend the General Body to agree to the alterations."

The present set of forms is thus adopted by both the Royal Institute of British Architects and the Institute of Builders. The debates on the various provisions of the Agreement* are well worth serious study.

Bibliographical Notes.

The following references are to a few articles or books relating to various matters affecting building agreements, and are given for the benefit of those desiring to make a more extended study of the subject:

* *Journal*, Royal Institute of British Architects, Vols. 2, 8 and 10.

"General Notes on Specifications and Contracts," by William B. Bamford, Assoc. M. Am. Soc. C. E. *The American Architect*, serial beginning Sept. 15th and ending Oct. 6th, 1909.

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"The Legal Position of Architects in Relation to Certificates and Awards," by J. A. Strahan. (A paper read before the Royal Institute of British Architects, Jan. 28th, 1895.) Abstract in *The Builder*, Vol. LXVIII, p. 82.

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"Engineers' Contracts and Specifications from a Contractor's Point of View," by James W. Rollins, Jr., M. Am. Soc. C. E. *Journal, Association of Engineering Societies*, July, 1907.

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"The Engineer in Construction Contracts," by John C. Wait, M. Am. Soc. C. E. *Transactions*, Association of Civil Engineers of Cornell University, 1902.

"Engineers' Specifications." (Observations on specifications in general and the legal effect of clauses of particular hardship to contractors.) *The Engineer* (London), Dec. 30th, 1904.

"The Influence of Specifications on Commercial Products," by C. B. Dudley, M. Am. Soc. C. E. (Presidential address before the American Society for Testing Materials.) *Proceedings*, American Society for Testing Materials, 1904. Reprint in *The Engineering Record*, July 9th, 1904.

"Arbitrations: A Text-Book for Arbitrators, Umpires, and All Connected with Arbitrations, more especially Architects, Engineers, and Surveyors, in Tabulated Form, with the Chief Cases Governing the Same, and an Appendix of Forms, Statutes, Rules, etc.," by Professor Bannister Fletcher, F. R. I. B. A., London (1904). Crown 8vo, 149 pages, cloth, 3d ed., revised and largely rewritten.

"Architects' Law Reports (illustrated): A Series of Revised Reports of Cases Decided in the Superior Courts, of Importance to Architects, Surveyors, and Others Interested in the Construction and Mainte-

nance of Buildings and Streets in London and the Provinces," compiled by Arthur Crow, F. R. I. B. A., Legal Editor; A. F. Jenkins, Barrister-at-law. London, Vol. I, Dec., 1904, 125 pages, 4to, paper cover; Vol. II, Dec., 1905, 275 pages, 4to, paper cover; Vol. III, Dec., 1908, 4to, paper cover.

"Building Cases: Being a Digest of Reported Decisions Affecting Architects, Surveyors, Builders, and Building Owners," by F. St. John Morrow, LL.D., Barrister-at-law. London (1906), Large 8vo.

"Building Contracts, Building Leases, and Building Statutes (English), with Precedents of Building Leases and Contracts, and Other Forms Connected with Building, and the Statute Law Relating to Building, with Notes and Cases under the Various Sections, Together with an Appendix of Unreported Building Cases," by Judge Emden. London (1907), 800 pages, 4th ed., revised, Large 8vo.

"Conditions of Contract," by Frank W. Macey. London, 1904.

"Engineering and Architectural Jurisprudence: A Presentation of the Law of Construction for Engineers, Architects, Contractors, Builders, Public Officials, and Attorneys-at-Law," by John C. Wait, M. Am. Soc. C. E. New York (1898), lxxx + 905 pages, 8vo, cloth.

"Law of Contracts," by John M. Brown (the author). Wash. (1907), 80 pages, 8vo, cloth.

"Statutory Provisions Relating to Government Contracts," by John M. Brown (the author). Wash. (1908), 49 pages, 8vo, cloth.

Divisions of Contracts.

Building contracts may be said to be divisible into four parts or divisions.

1.—Matters Preliminary to Contracts.—Being the terms under which a contract is awarded, and giving the general information necessary before the execution of the agreement. It would include the "Form of Agreement" and "Schedule of Conditions of Contract" as given hereafter.

2.—Agreement (or so-called Contract).—Being the instrument executed between the Owner and Contractor, legally and formally binding them to fulfill their obligations. It should properly include the "Schedule of Conditions of Contract."

3.—Specifications.—Being the detailed written description of the method of construction and of the quality and kind of materials to be furnished.

4.—Drawings.—Being the detailed, graphic design of the work to be done, with the quantity and sizes of materials.

This paper treats only of the Agreement (or so-called Contract).

The Agreement, preferably, should not contain any matters concerning the details of construction, these belonging properly to the specifications. There are, however, a number of legal requirements which could be included in the Agreement or in the Specification. Legally, they would be just as binding in one as in the other. These matters will be mentioned under the heading "General Conditions."

Reasons for Contracts.

A building contract is entered into by the Owner in order to obtain the services of a contractor who is skilled in that particular kind of work, and by the Contractor because he expects to make a certain profit in remuneration for his services and knowledge. Assured of his profit, it is immaterial to the Contractor what kind of work the Owner or Engineer may desire to have done, but if the Contractor's profit is not assured, he becomes vitally interested in anything that may tend to affect this prospective profit. With the Contractor, the work is purely a business proposition; he expects to make a certain profit for doing a particular amount of work. He must make a legitimate profit; his business life depends on it; and the more he feels this slipping away from him the more diligently will he find ways and means to reimburse himself. Even the best of contractors will try hard to obtain this profit in one way or another, else they must ultimately be forced from business and leave the field to those who are probably less competent contractors but shrewder business men.

A contract is intended to insure that an Owner will obtain a definite piece of work for a stated sum and within a given time, and to insure that, for doing this, a Contractor will be paid stated sums of money at definite times. The contract, therefore, should be drawn so that the Owner can demand and obtain his work and the Contractor his payments with the least delay and difficulty.

It is thought that the provisions of the Agreement and Schedule of Conditions of Contract given hereafter will protect more equitably the interests of all parties, and permit the Engineer to enforce more certainly the proper fulfillment of the contract obligations than has been possible under the majority of such documents which have come to the writer's notice.

Systems of Contracting.

At present the systems of contracting used in building work may be divided into two general classes, the lump-sum system and the various percentage systems. They are often designated by the manner in which the work is to be paid for, as:

1.—Lump sum;

2.—Cost, plus a percentage;

3.—Cost, plus a percentage—where the General Contractor takes estimates for sub-branches of work, and makes lump-sum contracts, with the Engineer's approval;

4.—Cost, plus a percentage, with guaranteed limit;

5.—Cost, plus a fixed sum;

6.—Cost, plus a fixed sum, with a bonus if cost is less than a fixed sum, and a penalty if it exceeds such sum.

Of these, the lump-sum system is the most usual, and is theoretically the best. Such a system pre-supposes that the contract defines the work to be performed, and that a contractor, therefore, can determine accurately what the execution of such work will cost. If, however, the contract requirements are indefinite, the Contractor is never certain, no matter how conscientious he may be, that he may not be required, under broad or ambiguous clauses of the contract, to furnish more than he honestly understood would be required from his reading of the contract when estimating. Thus he may be forced to do work at a net loss instead of a profit. Under the various percentage systems, on the contrary, the Contractor is assured of his profit, no matter how much work he may be required to do.

The lump-sum contract is superior to the percentage type because it more truly fosters fair and open competition, and is more economical, as has been demonstrated amply by many men of ripe experience. This presumes that the drawings and specifications describe completely and accurately everything that the Contractor must do or furnish, especially where an expense is involved. If this is not done, the Contractor's prices become mere gambles on probabilities, and not accurate estimates of actual costs plus a legitimate profit. In such a case is it to be wondered at that a Contractor will fight at every legitimate opening in order to protect his final profit? He is not always able legally to demand additional compensation for the work which

has actually cost him more than the estimated amount, therefore his only opportunity to make up his losses and come out with a fair total profit is by obtaining good prices for additional work, or, if it does not appear that there will be any such worth while, the Contractor must find flaws in the contract and endeavor to get all the extra money he can, through technicalities or otherwise.

With the irresponsible or dishonest Contractor there need be no sympathy, but by far the greater number of Contractors only desire their legitimate profit, and the prospect of making a fair profit by hard, honest work is none too bright at the present time, under the usual lump-sum contract.

Contractors have found that the easiest and most practical way to overcome the disadvantages of the lump-sum estimates is by the various percentage systems. The Contractor is thus assured of his profits, and the Owner and Engineer can change or increase the work to their hearts' content, as all additional expense involved is borne by the Owner.

Assuming that the Contractor is honest, and that the bills and statements he presents for payment are correct, and have all discounts, except a cash discount deducted, the cost of percentage work is apt to exceed the cost of work done under a lump-sum contract. The reasons for most of this increase are not hard to find. The Contractor, not being responsible for the cost, and having his profits assured, readily makes any changes and does work as directed, no matter how expensive or uneconomical such work may be, and the Owner does not realize how expensive his fads or fancies will be until the work is completed. Then again, if the percentage is based on the total cost, there is not the incentive for the Contractor to become as zealous in obtaining the closest prices for materials as he would under a lump-sum contract.

Another most serious objection to percentage work is due to the fact that mechanics are inclined to loaf, even under the most careful supervision, and it is hard to get as much work out of them as under the lump-sum system. They seem to feel that as their employer, the Contractor, will not suffer any loss, there will be no harm done, or they will not be in danger of losing their positions.

Many of the most serious objections to the percentage system have been overcome in varying degrees under Systems 3, 4, 5, and 6. These systems have much to commend them, and, with all their faults, they

will eventually supersede the lump-sum system, unless more care is taken to make contracts complete and accurate.

The English method of "Quantity Surveying" is another excellent attempt to overcome the worst objections to the lump-sum system of estimating, but, as at present in use, it would be too cumbersome for American practice. There the Owner or Engineer has the work to be done itemized in detail, and on this itemized statement, submitted to all Contractors, they fill in their prices. These lists go into matters in minute detail, and, as all Contractors estimate on the same list, the advantage is obvious. The Royal Institute of British Architects had approved forms, which are only slight modifications of the one for lump-sum contracting, in which the principal change is in making the "Quantities" a part of the contract. Under such a contract, the Contractor would be able to obtain compensation for all items not mentioned specifically in the "Schedule of Quantities."

Practical and Equitable Contracts.

Many contracts appear to be drawn by the Engineer solely on the assumption that the Owner's interests must be most carefully guarded against failure by the Contractor to fulfill his agreements properly. In the past, when profits were sufficiently liberal, Contractors could afford to take chances that "club-clauses" would not be strictly enforced, and profits lost by unfair clauses could be made up on "extras." Today, however, competition is sharper, and profits are too often figured on too slender a margin to permit a Contractor to "take care" of losses caused by unfair clauses. No wonder, then, that the Contractor becomes more and more technical and troublesome over small omissions or errors, whereas, before, he could charge these minor matters to profit and loss, or good-will. The chances which a Contractor takes that an Owner will not be fair-minded should not be overlooked. An Owner may become very exacting, and may insist on "strict and literal" compliance with the letter of the specifications and contract, and may place an Engineer in a very embarrassing position by referring to and insisting on the literal fulfillment of certain ambiguous conditions or the "taking of every pound of flesh," against the Engineer's better judgment.

The value of a just contract to the Owner and the Engineer is as great as its value to the Contractor. A just contract, provided it is

accurate and free from ambiguity, cheapens the cost of work by eliminating the necessity for sums to cover possible contingencies; it broadens real competition, and encourages lower estimates; it ensures more rapid erection, and also acts as a powerful regulator of the mutual understanding and confidence in the fairness of all parties. Fairness, like oil for machinery, is necessary to prevent friction.

The Agreement is only one of the divisions of a contract. It is the specifications and drawings, together, which must give the Contractor a clear and complete knowledge of the work he must perform and the materials he must furnish, and should contain all the information necessary to permit him to make an accurate estimate and to carry out the work properly. The specification requirements should define clearly the result, the accomplishment of which is the object of the contract. It is a general statement of the work to be performed, a description of the materials, the quality required, and the workmanship to be performed, with definite limits as to what tests must be made in order to determine compliance with the requirements of the contract, or what defects would be sufficient cause for rejection.

Specification writing cannot be acquired successfully without practice and without broad knowledge, practical experience, and a careful study of the various materials and methods of construction and their relative costs. As ideas and methods change and improve, as a result of experience, so specifications should be changed to keep abreast of the times. An inexperienced writer is very apt to be uneven in his descriptions, writing at unnecessary length upon matters with which he is familiar, and covering his ignorance of equally important matters by the brief and ambiguous direction that "the work shall be done to the entire satisfaction of the Engineer," "whose decision is to be final and binding on all parties." These statements are too often an admission that the Engineer did not know what he desired, or that the Owner had failed to express a preference and, therefore, had been left with unlimited future choice.

As previously stated, the ultimate object of a contract is to insure to the Owner that he will obtain a definite finished product, of a certain kind, for a particular cost, and in a given time; and to insure to the Contractor that he will obtain certain sums of money at definite times for doing a definite amount of work; and the Engineer is looked upon as the arbitrator or judge, to see that both parties receive their full

justice under the contract. Contracts, therefore, should be drawn so that the Owner can demand and obtain his work and the Contractor his payments with the least delay and difficulty.

Engineers have always claimed to be arbitrators between Owner and Contractor, and not the representative solely of one party. This position is a proper one, but to-day it is more of a myth than a fact. The anomalous position is brought about, not because Engineers are desirous of being unfair, but because of the agreements (or contracts) generally used, which make it extremely hard, if not almost impossible, for Engineers to be fair if they try to conform to the agreement provisions.

It might be claimed that one of the most practical benefits to Engineers and Architects, in the more general use of an agreement similar to the one herein suggested, might be that it would not so insidiously "lead us into temptation" but would "deliver us" from many of the pitfalls lurking unsuspectingly around many present contracts.

While there is abundant room and great need for material improvement in specifications, it must be recognized that, with the limitations to the best of human endeavors, it will be extremely difficult, even for the most conscientious and painstaking, to prepare specifications for large work which may not contain some unfair requirements, when viewed from the Contractor's standpoint. We can come near to the goal, certainly much nearer than is at present the practice, but possibly our slight oversight may mean a Contractor's heavy loss.

The writer, therefore, strongly feels that one of the most practical steps toward the realization of contracts which will be fair and just to all parties will be made by the more general acceptance of the principles contained in the following forms of Agreement and Schedule of Conditions of Contract, and the adoption of similar provisions in contracts.

It is suggested that the following Agreement and Schedule of Conditions of Contract, be read through first as a whole, in order to obtain a general knowledge of their scope, omitting the reading of the notes following. They should then be read, article by article, with the comments after each.

AGREEMENT.

(For Lump-Sum Contracts.)

This Agreement, made the ..... day of ....., in the year One Thousand and Nine Hundred and ....., by and between

....., of ....., hereinafter called the Contractor, and ..... of ....., hereinafter called the Owner.

Witnesseth, that the Contractor and Owner, for the considerations hereinafter named, agree as follows:

Article I.—The Contractor agrees to provide all materials and perform all works for the.....
(here briefly describe works)
.....
.....
of a..... at .....
(here briefly describe type of building) (here briefly describe location)
shown on drawings numbered ..... and as described in the Specifications marked....., prepared by ....., hereinafter called the Engineer; and to do all things required of him by the Drawings and Specifications and subject to the Conditions set forth in the Schedule hereto annexed (hereinafter referred to as "the said Conditions").

Article II.—The Contractor agrees to complete the works by and at the time or times hereinafter stated, to wit:.....
.....
.....

Liquidated damages should be a matter for special consideration in each separate contract. Many Engineers and Owners seem to desire them, possibly for the moral effect on Contractors. The history of the legality of such provisions should cause them to be used with caution.

In many cases the following clause might be as far as it would be advisable to go toward obtaining just and reasonable damages for delay in completion:

"The Contractor agrees that the time for completion of the works shall be considered as of the essence of the contract, and he agrees that for liquidated damages he will pay the Owner for the cost of all extra inspection, and for all amounts paid for rents, or for more protracted services on the part of the Engineer, or Clerk of the Works, or both, and other expenses entailed by the Owner by delay in completing the contract, and the Owner shall be entitled to retain from the amount otherwise to become due to the Contractor an amount sufficient to reimburse properly the Owner for such expenses. Provided, however, that the sum collected as aforesaid for liquidated damages shall not exceed the sum of $..... for each and every day's delay not caused by the Owner, or otherwise as provided in the said Conditions, Section XXIV (subject to Arbitration)."

Article III.—The Owner agrees to pay the Contractor for the execution of the contract ..... dollars ($.....) in current funds subject to additions and deductions as provided in the said Conditions, and upon the presentation of the certificates of the Engineer, as follows:

Before the ..... day of each month, the Contractor may submit to the Engineer a written statement showing (*a*) the value of the work and materials actually wrought into the building to the first of the month, (*b*) the value of materials delivered at the building but not incorporated into the work, and shall deduct the aggregate of previous payments. On or before the ..... day of the same month, the Engineer shall issue a certificate for .....% of the value of work as executed in the building, until the balance retained by the Owner amounts to the sum of $....., after which time the payments shall be for the full value of the work subsequently executed. The Contractor shall be entitled, under the certificate to be issued by the Engineer, to receive payment of ....., being a part of the said sum retained in hand when the works are practically completed, and in like manner to payment of the balance within a further period of ..... months, or as soon after the expiration of such period of ..... months as the work shall have been finally completed, and all defects made good according to the true intent and meaning hereof, whichever shall last happen. The Engineer, however, shall have authority to retain out of any monies actually due the Contractor only such sums as are provided by the said Conditions, Section XXX.

The manner of payment could be varied in many ways, as the conditions of the particular case made it necessary. The principle underlying the form suggested above, however, is important.

As before stated, it is as essential that the Contractor shall be able to demand and obtain his payments as that the Owner shall be able to obtain the work for which he pays.

In Article III the Contractor's right to definite payments is expressed clearly. The Engineer must issue a certificate for money due, and the Owner must pay the amount due. It should be noted that the Engineer is allowed ample discretionary power to withhold any money not justly due, as provided by Section XXX of the Conditions. Thus the obligations become fairly clean cut. If the Contractor's work is not done properly, there is ample power to withhold money to cover definite defects. On the other hand, if the Engineer or Owner withhold any payments without just cause, the Contractor has recourse to

Arbitration, under Section XXXIV of the Conditions, or to terminate the contract, under certain contingencies, as provided in Section XXXIII.

The idea of retaining percentages until the completion of the work is intended to insure its proper completion. The usual American practice of retaining an unchanging percentage from start to finish is unnecessarily extravagant, and in contracts involving large amounts is more harmful than beneficial. After the work has been approximately half completed, the remainder should be paid for in full without retaining a percentage. Of course, additional sums can be retained to cover known defects. The sum retained when the work is half completed is usually sufficiently liberal to cover possible contingencies, and is just as effectual as to continue to increase it until completion.

Article IV.—It is mutually agreed that the term "the Engineer" in the said Conditions shall mean the said .....or in the event of his death or ceasing to be the Engineer, for the purpose of the contract, such other person as shall be nominated for that purpose by the Owner, not being a person to whom the Contractor shall object for reasons considered to be sufficient by the Arbitrators or Arbitrator mentioned in the said Conditions. Provided, always, that no person subsequently appointed to be Engineer under this contract shall be entitled to disregard or overrule any decision, or approval, or direction, given or expressed by the Engineer for the time being.

The purpose of Article IV is to cover cases which may be unusual but are generally very troublesome when they occur. It would prevent a subsequent Engineer from altering at will matters of personal opinion or choice of his predecessor. No two minds are exactly alike, and few Engineers or Architects have exactly similar tastes. Should an Engineer desire to change work which may be purely matters of opinion or judgment of both himself and his predecessor, such changes should not be paid for by the Contractor, but by the Owner, if he decides to follow the suggestions of his adviser. Of course, defective work should be condemned at any time, when discovered. Should an Engineer desire to change some orders of his predecessor, and take the ground that certain work should be done as a part of the contract, he should not have arbitrary power to enforce his will, but could order the work done as provided by Section XVII of the Conditions, leaving to the

Contractor the right of appeal to arbitration, under Section XXXIV, if he desires.

Article V.—It is mutually agreed that the said Conditions shall be read and construed as forming a part of this Agreement, and the parties hereto will respectively abide by and submit themselves to the conditions and stipulations, and perform the agreements on their parts, respectively, in such Conditions contained.

Article VI.—The Contractor and Owner, for themselves, their successors, executors, administrators, and assigns, hereby agree to the full performance of the covenants and agreements herein contained.

In Witness Whereof, the Contractor and Owner have hereunto set their hands and seals, the day and year first above written.

In presence of

.... [SEAL]
.... [SEAL]

SCHEDULE OF CONDITIONS OF CONTRACT.

I.—Drawings and Specifications.—The works shall be erected in accordance with the directions and to the reasonable satisfaction of the Engineer, in accordance with the Contract Drawings and Specifications, and in accordance with such further drawings, details, instructions, directions, and explanations as may from time to time be given by the Engineer. If the work shown on any such further drawings or details, or explanations be, in the opinion of the Contractor, extra to that comprised in the Contract, he shall, before proceeding with such work, give notice in writing to this effect to the Engineer. In the event of the Engineer and Contractor failing to agree as to whether or not there is any extra, and of the Engineer deciding that the Contractor is to carry out the said work, the Contractor shall accordingly do so, and the question whether or not there is any extra, and if so the amount thereof, shall, failing agreement, be settled by Arbitration, as provided in Section XXXIV, and the Contractor will be paid accordingly.

This section covers the troublesome question of work shown by details, etc., being in excess of the contract requirements. It might be reasonable to assume that the Engineer would know better than anyone else whether his details require extra work, and the Contractor should be permitted at any time to recover the value of such work upon proving that it was extra to the contract. There are many sound objections to such a course, and the more satisfactory of the two evils would seem to be to require the Contractor to notify the Engineer

of any extra work, before proceeding with it, as a precedent to obtaining extra compensation.

II.—Contractor to Provide Everything Necessary.—The Contractor shall provide everything necessary for the proper execution of the works, within the limits established by the Drawings and Specifications, according to the true intent and meaning of the Drawings and Specifications taken together, whether the same may or may not be particularly shown on the Drawings or described in the Specifications, provided that the same is reasonably to be inferred therefrom; and if the Contractor finds any discrepancy in the Drawings, or between the Drawings and Specifications, he shall immediately refer the same to the Engineer, who shall decide which shall be followed. Figured dimensions shall be followed in preference to the scale, and figures on large-scale drawings shall take precedence over those at smaller scale.

III.—Copies of Drawings and Specifications.—The Contract Drawings and Specifications shall remain in the custody of the Engineer, and shall be produced by him at his office as and when required by the Contractor or the Owner. Three complete copies of all Drawings and of the Specifications shall be furnished by the Engineer, free of cost, to the Contractor, for his own use. The Engineer shall furnish to the Contractor within ..... days after the receipt by him of a request for the same, any details which, in the opinion of the Engineer, are necessary for the execution of any part of the work, such request to be made only within a reasonable time before it is necessary to execute such work in order to fulfill the contract. A complete set of such copies and details shall be kept on the works until the completion thereof, and the Engineer or his representative shall at all reasonable times have access to the same, and, as instruments of service, all drawings are the property of the Engineer and shall be returned to the Engineer by the Contractor at the time of making the final payment under the contract.

IV.—Conform to Laws, Regulations, Etc.—The Contractor shall conform to the provisions of all the State building and sanitary laws, and with all municipal or local ordinances, rules, and regulations relating to building and to the preservation of the public health and safety, also with all regulations of the Board of Fire Underwriters and Fire Insurance Authorities having jurisdiction, and of any Water and Lighting Companies with whose systems the structure is proposed to be connected, and, before making any variations from the Drawings and Specifications that may be necessary by so conforming, shall give to the Engineer written notice, specifying the variation proposed to be made, and the reason for making it, and apply for instructions thereon. In case the Contractor shall not in due course receive such instructions,

he shall proceed with the work, conforming to the provisions, ordinances, rules, and regulations in question, and the variation thus necessitated shall be dealt with under Section XXXIV. All contract requirements over, above, and beyond the said provisions, ordinances, rules, and regulations, shall be fully complied with. The Contractor, at his own expense, unless the law specifically requires the Owner to do so, shall obtain all necessary permits and licenses, give all necessary notices, and pay all legal fees.

The laws, ordinances, etc., of States and municipalities are often very divergent in their requirements, and in places positively contradictory. The meaning of many of their requirements, together with a great many of those of various Fire Insurance Authorities, is not easy to determine by the most careful reading. Then again, the local authorities may have discretionary powers, and their "personal preferences" may be difficult to obtain, even by local men. Thus it would appear that, if an Engineer cannot interpret correctly the local requirements in his drawings and specifications, it is unfair to place the responsibility, by a blanket clause, on the Contractor. If, therefore, the authorities require work to be done in a different manner than that called for by the contract documents, and such work involves additional expense, the Owner, and not the Contractor, should pay for the same.

V.—Setting Out Work.—The Contractor shall set out the work in accordance with the Owner's survey and the Engineer's drawings, and, during the progress of the building, shall correct at his own expense any errors arising from inaccurate setting out, unless the Engineer shall decide to the contrary.

It is advisable that the Owner have a survey made, and guaranteed, if desired, by some recognized Title Guaranty Company. With this survey and the drawings, the Contractor should be held to lay out the work correctly; but, to renounce all responsibility for the survey or drawings, and endeavor to place the responsibility for all conditions, both known and unknown, on a Contractor, is not good business policy, nor keen professional foresight, though there may be specific cases apparently to justify such a course.

VI.—Foreman.—The Contractor shall keep constantly on the works a competent general foreman, and any directions or explanations given by the Engineer to such foreman shall be held to have been given to the Contractor.

VII.—Assignment of Contract.—The Contractor shall not, without the written consent of the Engineer, assign this agreement, and, before subletting any portion of the works, shall notify the Engineer of such fact, in writing. No sub-contractors, to whom the Engineer has made written objections, shall be employed, and the Contractor shall assume full responsibility for all his sub-contractors.

Sub-contractors are not legally recognized as parties to a contract, but, in many cases the power is assumed of dictating to a Contractor what persons he must employ as sub-contractors. Such a course is not only unfair, but might cause legal difficulties later, should the Contractor take the ground that he is not responsible for the acts of a sub-contractor, not of his own selection, but who was forced upon him by the Engineer or Owner. In the foregoing section the Engineer may disapprove of sub-contractors who are not satisfactory, but he is not permitted to dictate who must be employed as sub-contractors. Where the Engineer approves a sub-contractor, it can be implied that he considers that the person whom he may approve will be satisfactory, while, by confining himself to the disapproval of unsatisfactory sub-contractors, no such satisfaction can be implied, as the persons finally employed may be entirely unknown to the Engineer.

VIII.—Work by Other Contractors.—The Contractor shall permit the execution of work by any other mechanics or trades employed by the Owner, and shall afford them every reasonable facility for the proper erection of their work. The Owner agrees not to employ any other mechanics or trades on the work against whom the Contractor shall make what the Engineer considers a reasonable objection, or who will not enter into a contract with the Contractor guaranteeing the due performance of his work and indemnifying the Contractor against valid claims which may arise during the performance of the work.

The object of this section is simply to minimize the chances for disagreements between independent contractors and their making claims to the Owner for damages done to them by other contractors. By this section, if a General Contractor is responsible for the completion of certain work, he is permitted to make reasonable objection to other independent Contractors before they are employed, and to exact a warranty for valid claims arising out of the work. If they agree to indemnify each other, the Owner has then fulfilled his obligations; then any disagreements must be settled between the individual Con-

tractors, and the Owner can demand with a clear conscience that they shall perform their contract obligations, and settle their own differences.

IX.—Materials, Etc., to Conform to Specifications.—All materials and workmanship shall be of the respective kinds described in the Specifications, and the Contractor, upon the request of the Engineer, shall furnish him with vouchers to prove that the materials are such as are specified. The Contractor shall also furnish duplicate labelled samples of materials and workmanship, for the Engineer's approval, and the materials and workmanship furnished shall be equal to the approved samples.

X.—Dismissal of Workmen.—The Contractor, on the written request of the Engineer, shall immediately dismiss from the works any person employed thereon by him, who, in the opinion of the Engineer, may be incompetent, or who misconducts himself, and such person shall not be again employed on the works without the permission of the Engineer.

XI.—Access to Work.—The Engineer and any person authorized by him shall at all reasonable times have access to the works, and the Engineer and his representatives shall at like times have access to the workshops of the Contractor or other places where work is being prepared for the building.

XII.—Clerk of Works.—The Clerk of the Works or other representative shall be considered to act solely as inspector, and under the Engineer, and the Contractor shall afford him every facility for examining the works and materials. The Clerk of the Works or other representative shall have no power to revoke, alter, enlarge, or relax any requirements of the contract, except in so far as such authority may be specifically conferred by a written order of the Engineer.

XIII.—Work to Be Opened Up.—The Contractor, at the request of the Engineer, and within such times as the Engineer shall name, shall open for inspection any work covered up; and should the Contractor refuse or neglect to comply with such request, the Engineer may employ other workmen to open up the same. If the said work has been covered up in contravention of the Engineer's instructions, or if, on being opened up, it is found to be not in accordance with the Drawings and Specifications or the instructions of the Engineer, the expense of opening and covering it up again, whether done by the Contractor or such other workmen, shall be borne by, and shall be recoverable from, the Contractor, or may be deducted, as hereafter provided. If the work has not been covered up in contravention of such instructions, and be found in accordance with the said Drawings and Specifications or instructions, then the expense aforesaid shall be borne by the Owner and be added to the contract sum: provided always that in the case of foundations, or other urgent work thus opened up and requiring immediate attention,

the Engineer, within a reasonable time after the receipt of notice from the Contractor that the work has been so opened, shall make or cause the inspection thereof to be made, and at the expiration of such time, if such inspection shall not have been made, the Contractor may cover up the same, and shall not be required to open it up again for inspection except at the expense of the Owner.

This section seems to indicate the fairest practical way of solving a perplexing problem.

XIV.—Removal of Work.—During the progress of the works, the Engineer shall have power to order in writing from time to time the removal from the works, within such reasonable time or times as may be specified in the order, of any materials which in the opinion of the Engineer are not in accordance with the Specifications or the instructions of the Engineer, the substitution of proper materials, and the removal and proper re-execution of any work executed with materials or workmanship not in accordance with the Drawings and Specifications or instructions; and the Contractor shall forthwith carry out such order at his own cost. In case of default of the Contractor to carry out such order, the Owner shall have power to employ and pay other persons to carry out the same; and all expenses consequent thereon or incidental thereto shall be borne by the Contractor, and shall be recoverable from him by the Owner, or may be deducted by the Owner from any monies due or that may become due to the Contractor. If, in the opinion of the Engineer, it is not expedient to remove materials delivered or alter work done which is not in accordance with the contract, the Owner may deduct the difference in value of the materials as delivered and the work as done and that required by the Drawings and Specifications, together with a fair amount for damages, which sum shall be determined by the Engineer (subject to Arbitration).

XV.—Defective Work.—Any defects, shrinkage, or other faults which may appear within.....months from the completion of the works, and, in the opinion of the Engineer, arising from materials or workmanship not in accordance with the Drawings and Specifications or the instructions of the Engineer, or any damage to pointing by frost appearing within the like period, shall, upon the direction in writing of the Engineer, and within such reasonable time as shall be specified therein, be amended and made good by the Contractor at his own expense, unless the Engineer shall decide that he ought to be paid for the same; and, in case of default, the Owner may employ and pay other persons to amend and make good such defects, shrinkage, or other faults or damage, and all expenses consequent thereon or incidental thereto shall be borne by the Contractor and shall be recoverable from him by the Owner from any monies due or that may become due to the Contractor. Should any defective work have been done

or should any defective materials be supplied by any sub-contractor employed on the work who has been nominated or approved by the Engineer, as provided in Sections VII and XXIX, the Contractor shall be liable to make good in the same manner as if such work or materials had been done or supplied by the Contractor and had been subject to the provisions of this and the preceding Section.

XVI.—Copy of Quantities, Etc.—The Contractor, if required before executing this agreement or before making application for any payment, shall furnish to the Engineer, upon a blank form prepared by him, a correct, verified, itemized statement, showing the quantities of labor and materials that will be required to perform the principal items of the contract, together with their prices; and the amount included in his estimate for each general item of work to be furnished complete and in position, including in each item its due proportion of expense and profit, the sum of the different items equalling the total amount of the proposal. The above schedule shall be for the use of the Engineer, at his discretion, only in preparing estimates for payments on account to the Contractor.

If this section is read in connection with Sections XVII and XVIII, it will be seen that possibly many apparently desirable modifications are covered by the provisions of those two sections.

XVII.—Variations and Extras.—The Contractor, when authorized by the Engineer, or as provided by Section IV, shall vary by altering, adding to, or deducting from the contract requirements; such authorization is to be sufficiently proven by any writing or drawing signed by the Engineer, or by any subsequent written approval by him, but the Contractor shall make no variation without such authorization. No claim for extra compensation will be allowed unless it shall have been executed under the provision of Section IV, or by the authority of the Engineer, as herein mentioned. Such an extra is hereinafter referred to as an authorized extra.

Variations in the contract (or extras) is one of the most troublesome phases of lump-sum contract work. By the provisions of this section and Section XVIII, with the final redress to arbitration under Section XXXIV, it is hoped, that the greater part of such controversies can be adjusted satisfactorily.

XVIII.—Price for Extras.—No variation shall vitiate the contract; but all alterations, additions to, or deductions from the contract requirements, which may have been made with the authority of the Engineer, or without his authority, provided he gives subsequently a written sanction to such omission, shall be valued in one of the four following ways, as may be selected by the Owner:

A.—By Estimate and Acceptance of a Lump Sum. The Contractor shall submit an estimate for making the variation contemplated, and in case an agreement as to price cannot be reached, the Engineer may order the work to proceed and the Contractor shall forthwith proceed with such variation and leave the price to be settled by Arbitration. A tentative amount, however, will be allowed by the Owner, equal to the proportion of the amount of the estimate conceded by both the Owner and Contractor, and payments will be made on the basis of such amount "without prejudice" until the correct amount shall be determined by Arbitration.

B.—By Unit Prices. Where the Contractor shall have given, in his estimate for the contract, or in subsequent accepted estimates, unit prices for specified parts of the work, the Engineer may make an award of a sum based on these unit prices, subject to Arbitration.

C.—By Net Cost. In this case the Contractor will be paid the actual net amount paid by him for materials; after all trade discounts, except a *bona fide* cash discount, have been deducted; plus the cost of labor or material required to erect and finish the work properly; plus ....% for profit, general office superintendence, and other general office expenses not peculiar to this contract; plus ....% per month on the actual cost for reproducing the plant as used for such extra work, for plant rental. The total net cost shall not exceed the usual reasonable cost for similar work, unless the Contractor can prove conclusively that the excess was due to exceptional and unavoidable difficulties or conditions. At required intervals, of not less than seven days, the Contractor shall render detailed statements and vouchers, and the Engineer will award an amount as the net cost, subject to Arbitration. In variations involving the omission of work, the amount to be deducted for such omission shall be based on the net cost as given above, plus ....% for prospective profit.

D.—By Cost Plus a Lump Sum. In such cases the Contractor shall submit an estimate which he agrees the variation shall not exceed, plus a sum for profit. The Contractor, at required intervals, of not less than seven days, shall render detailed statements and vouchers of the net cost, on the same basis as given under *"C"* above, except that no profit shall be included. If the actual cost is less than that given in his estimate, the Contractor shall receive, in addition to the lump sum profit, ...% of the difference between the actual cost and the amount of his estimate. If the actual cost is greater than that given in his estimate, the Contractor shall receive, in addition to the amount of his estimate, only ....% of the difference between the actual cost and the amount of his estimate.

The foregoing methods of determining the price to be paid for variations to the contract (or extras) with the redress to Arbitration

(Section XXXIV), is in principle the most equitable method that has come to the writer's notice, for solving one of the most vital parts of a contract—the proper compensation for legitimate extra work. This section, in detail, is entirely different from anything which the writer has seen in use, and he suggests for it special consideration. In principle, it is similar to the requirements of the Royal Institute of British Architects.

XIX.—Damage to Persons and Property.—The Contractor shall indemnify and save harmless the Owner from any and all claims and demands which may be made by reason of (1) any injury or damage suffered or sustained by any person or corporation, caused by, or alleged to have been caused by, any act or omission of the Contractor or his agents, servants, workmen, employees, or sub-contractors, in the course of the performance of the work; and (2) any injury or damage suffered or sustained by any such agents, servants, workmen, employees, or sub-contractors, however caused, in or about said building; and the Contractor, at his own cost, expense, and risk, shall defend any and all actions, suits, or other legal proceeding that may be brought or instituted against the Owner on any such claim or demand, and pay and satisfy any judgment that may be rendered against the Owner in any such action, suit, or other legal proceeding, or result thereof; provided, however, that this section shall not be construed as an assumption by the Contractor of any liability to the Owner for any injury or damage caused by an employee of the Owner who is not a Contractor or sub-contractor, or an agent, servant, workman, or employee of a Contractor or sub-contractor engaged in the performance of work in or about said building.

XX.—Damage to Work.—The Contractor shall assume the responsibility for, and repair at his own expense, all damage to his work, or the work of other Contractors, or to the building and contents, caused by the installation of work or by the carelessness or negligence of himself, his sub-contractors, or employees. He shall also be responsible for, and make good, any damage to adjoining property, caused by any movement or settlement, or by material or implements falling upon the same during construction, or for any other cause for which the Owner may be held liable, and shall thoroughly complete the whole of the works.

XXI.—Patent Infringement, Etc.—The Contractor shall indemnify and save harmless the Owner from any and all claims and demands which may be made by reason of any infringement, or alleged infringement, of any patent, patent rights, or claim, caused, or alleged to have been caused, by the use of any apparatus or appliances, or portions thereof, furnished or installed by the Contractor; and the Contractor,

at his own cost, expense, and risk, shall defend any and all actions, suits, or other legal proceedings that may be brought or instituted against the owner on any such claim or demand, and pay and satisfy any judgment or decree that may be rendered or adjudged against the Owner in any action, suit, or other legal proceeding or result thereof.

XXII.—Insurance.—During the progress of the works, the Owner will maintain insurance on said works, in his own name and in the name of the Contractor, against loss or damage by fire or lightning, and the policies will cover all work incorporated in the building, except that of excavation and foundations below the level of the ground, and all materials for same in or about the premises, and be made payable to the Owner, for adjustment between the parties hereto, as their interests may appear. At the end of each month the Contractor shall deliver to the Owner a statement showing the total value of work done and materials delivered, as herein described, to that date, and the owner will maintain said insurance in accordance with the latest statement thus delivered.

The objection to most insurance clauses is the difficulty of adjusting properly the interests as they may appear. Fire Insurance Companies usually hold such payments until both parties sign an agreement stating the proportionate amounts each party is to receive. This adjusting, in case of a disagreement, may consume considerable time, and delay the continuance of the work. Under Section XXII the amount of the insurance is payable directly to the Owner, and as he receives monthly a statement from the Contractor and insures the Contractor in accordance with such statement, it is fairly simple to estimate the Contractor's loss at any time. If the Owner neglects to insure himself properly, he then has no one but himself to blame.

XXIII.—Damages for Delay. Should the Owner claim damages for delay in the completion of the works, the Engineer, if, in his opinion, any damages be payable, will make an award on the basis stated in the Agreement, and will write the amount across the face of the final certificate as "the amount to be deducted from the face hereof for delay in completion of the work" (subject to Arbitration). Thereupon the Engineer's services in this matter shall terminate, except as provided under Section XXXIV.

Should the Contractor, by his own fault or negligence, unnecessarily delay the completion of the work, thereby necessitating unusual or more protracted services on the part of the Engineer or the Clerk of the Works, or both, the Owner shall be entitled to retain from the amount otherwise to become due the Contractor an amount sufficient

to reimburse properly either the Engineer or the Clerk of the Works, or both, for such protracted or unusual services (subject to Arbitration).

This section should be read in connection with any damages to be allowed under Article II of the Agreement. It should be distinctly stated in the Agreement whether or not the specified sum for damages includes the protracted services of the Engineer, etc., herein mentioned. In many cases compensation for such protracted services might be all the damage that should justly be demanded.

XXIV.—Extension of Time.—If, in the opinion of the Engineer, the work be delayed by *force majeure* or by reason of any exceptionally inclement weather, or by reason of instructions from the Engineer in consequence of proceedings taken or threatened by, or disputes with, adjoining or neighboring owners, or by the work or delay of other Contractors or mechanics engaged or nominated by the Owner or the Engineer, and not referred to in the Specifications, or by reason of authorized extras or additions, or in consequence of any notice reasonably given by the Contractor in pursuance of Section IV, or by the combined action of workmen, in no wise caused by or resulting from default or collusion on the part of the Contractor, or in consequence of the Contractor not having received in due time necessary instructions from the Engineer for which he shall have specifically applied in writing, the Engineer will make a fair and reasonable extension of time for completion in respect thereof. In case of such combined action of workmen, the Contractor shall, as soon as may be, give to the Engineer written notice thereof; but the Contractor shall, nevertheless, use his best endeavors to prevent delay, and shall do all that may reasonably be required, to the satisfaction of the Engineer, to proceed with the works.

The usual practice is to grant an extension of time to a Contractor equal to the time which he may be delayed, provided he gives notice of such delay within 48 hours. The reasons for which extensions may be granted are usually very few, and not always of the most importance. The section suggested above will probably be objected to as not definitely specifying the length of the extension of time that may be allowed. The Engineer is empowered to make a fair and reasonable extension of time for completion, and he can thus take into account matters which it would be hard to cover by an arbitrary clause. Should either party feel aggrieved by the length of time of the extension, it has the right of appeal to Arbitration (Section XXXIV).

XXV.—Suspension of Work by Contractor.—If the Contractor, except on account of any legal restraint upon the Owner preventing the continuance of the works, or on account of any of the causes mentioned in Section XXIV, or in case of any certificate being withheld or not paid when due, shall suspend the works, or, in the opinion of the Engineer, shall neglect or fail to proceed with due diligence in the performance of his part of the contract, or if he shall more than once make default in the respects mentioned in Section XIV, the Owner, by the Engineer, shall have power to give notice in writing to the Contractor requiring that the works be proceeded with in a reasonable manner and with reasonable despatch. Such notice shall not be unreasonably or vexatiously given, and must signify that it purports to be a notice under the provisions of this section, and must specify the act or default on the part of the Contractor upon which it is based. After such notice shall have been given, the Contractor shall not be at liberty to remove from the site or works, or from any ground contiguous thereto, any plant or materials belonging to him which shall have been placed thereon for the purpose of the works; and the Owner shall have a lien upon all such plant and materials, to subsist from the date of such notice being given until the notice shall have been complied with. Provided always that such lien shall not, under any circumstances, subsist after the expiration of thirty-one days from the date of such notice being given, unless the Owner shall have entered upon and taken possession of the works and site, as hereinafter provided. If the Contractor shall fail for .... days after the time given in such notice to proceed with the works as therein prescribed, the Owner shall be at liberty to provide any such labor or materials, and to deduct the cost thereof from any money then due or thereafter to become due to the Contractor under the contract.

The general purport of this section is similar to the majority of similar American clauses. Its wording, however, is freer from uncertainties, and deserves careful attention.

XXVI.—Termination of Contract by Owner.—If the Contractor shall fail for .... days after such notice has been given as provided in Section XXV, to proceed with the work as therein prescribed, and if the Engineer shall then certify that such refusal, neglect, or failure is sufficient ground for such action, the Owner may enter upon and take possession of the works and site, and of all plant and materials thereon (or on any ground contiguous thereto), intended to be used for the works, and all such materials as above mentioned shall thereupon become the property of the Owner absolutely, and the Owner shall retain and hold a lien upon all such plant until the works shall have been completed under the powers hereinafter conferred on him. If the

Owner shall exercise the above power he may engage any other person to complete the works, and exclude the Contractor, his agents, and servants, from entry upon or access to the same, except that the Contractor or any one person nominated by him may have access at all reasonable times to inspect, survey, and measure the works. And the Owner shall take such steps as, in the opinion of the Engineer, may be reasonably necessary for completing the works without undue delay or expense, using for that purpose the plant and materials above mentioned, in so far as they are suitable and adapted to such use. Upon the completion of the works, the Engineer shall certify the amount of the expense properly incurred consequent on and incidental to the default of the Contractor as aforesaid, and in completing the works by other persons. Should the amount thus certified as the expenses properly incurred be less than the amount which would have been due to the Contractor upon the completion of the work by him, the difference will be paid to the Contractor by the Owner; should the amount of the former exceed the latter, the difference shall be paid by the Contractor to the Owner. The Owner will not be liable to make any further payment or compensation to the Contractor for or on account of the proper use of the plant for the completion of the works under the provisions hereinbefore contained other than such payments as are included in the contract price. After the works shall have been thus completed by persons other than the Contractor, under the provisions hereinbefore contained, the Owner shall give notice to the Contractor of such completion, and may require him from time to time, before and after completion, to remove his plant and all such materials as aforesaid as may not have been used in the completion of the works, from the site. If such plant and materials are not removed within a reasonable time after notice shall have been given, the Owner may remove and sell the same, holding the proceeds, less the cost of their removal and sale, to the credit of the Contractor. Any notice to be given to the Contractor under this clause shall be given by leaving the same at the place of business of the Contractor, or by a registered letter sent to him at that address.

This section conforms in substance to the generally accepted American practice, but its wording is more definite, and it leaves fewer loopholes for argument or controversy.

XXVII.—Net Cost.—The words, "Net Cost," or the initials, "N. C.," used in the Specifications, for materials to be obtained and installed by the Contractor, shall mean, unless otherwise stated in the Specifications, the sum paid to merchants, after deducting all trade discounts, for such materials, in the ordinary course of delivery, but not deducting discount for cash, and such sum shall be exclusive of special cartage, the cost of installation, and Contractor's profit.

XXVIII.—Provisional Sums.—The provisional sums mentioned in the Specifications for materials to be supplied or for work to be performed by special contractors or mechanics, or for other works or fittings to the building, shall be paid and expended at such times, and in such amounts, and to and in favor of such persons, as the Engineer shall direct, and sums thus expended shall be payable by the Contractor without deduction or discount, or (without prejudice to any rights of the Contractor existing under the contract referred to in Section XXIX) by the Owner to the said Contractors or mechanics. The value of works which are executed by the Contractor in respect of provisional sums, or in additional works, shall be ascertained as provided by Section XVIII. At the settlement of accounts the amount paid by the Contractor to the said contractors and mechanics, and the said value of such works executed by the Contractor, shall be set against all provisional sums or any sum provided for additional works, and the balance, after allowing *pro rata* for the Contractor's profits at the rate of ....%, unless different rates are contained in the Contractor's original estimate, shall be added to or deducted from the contract sum, provided that in estimating the amounts paid as last herein provided no deductions shall be made by or on behalf of the Owner in respect of any damages paid by the sub-contractor to the Contractor, the intention being that the Contractor and not the Owner shall have the benefit of any such damages.

There are many cases where important matters may not be definitely determined before the execution of a contract. The practice in many cases is to insert a blanket clause giving almost unlimited selection to either Owner or Engineer. In such case a Contractor must make a shrewd guess at the amount to include in his estimate. Or something as nearly as possible to what may be required may be specified, and afterward the problem is to reconcile the cost of the work as specified with the work as required. The idea of having Provisional Sums inserted in a contract to cover matters not distinctly determined, is not new, but the above section may suggest a wording which will overcome many of the previous objections to such Provisional Sum clauses.

XXIX.—Sub-Contractors.—All contractors, merchants, mechanics, or others executing any work, or supplying any materials for which net cost prices or provisional sums are included in the Specifications, who may at any time be nominated, selected, or approved by the Engineer are hereby declared to be sub-contractors employed by the Contractor; but no sub-contractor shall be employed upon the works against whom the Contractor shall make what the Engineer considers reasonable objections, or who shall not enter into a contract with the

Contractor upon terms and conditions consistent with those in this Contract, and securing the due performance and maintenance of the works supplied or executed by such sub-contractors, and indemnifying the Contractor against any claims arising out of the misuse by the sub-contractor or his workmen of any scaffold erected or plant used by the Contractor, or that may be made against the Contractor in consequence of any act, omission, or default of the sub-contractor, his servants or agents, and against any liability, as given under Sections XIX, XXI, or XXII.

This section is intended to cover cases where it becomes desirable to have special Contractors execute work covered by Provisional Sums. In many cases particular types or makes of articles cannot be obtained excepting from special parties. To require a Contractor to employ such parties and not be relieved from responsibility for their actions, is a difficult problem. This section will prevent a Contractor, who may be under heavy penalty to complete his work, from being compelled to employ a particular Contractor for a sub-branch, who may absolutely refuse to indemnify the Contractor if he fails to complete his work on contract time, or to give a warrant for the faithful performance of the work.

XXX.—Payment and Certificate.—The Engineer shall issue his certificates for payments due the Contractor in accordance with the terms of the Agreement, provided, however, that the Engineer may withhold only such proportion of any sum due as in his judgment will cover the expense of the liens as given under Section XXXI, or of correcting such violation as given in Section XIV, and such payments shall not become due until violations are properly corrected (subject to Arbitration) or the liens are removed as hereinafter provided. No certificate given or payment made under the contract, nor partial or entire occupancy of the building by the Owner, shall be construed as an acceptance of defective work, or of improper materials, or as condoning any omission, provided, however, that in the case of a partial or entire occupancy of the building by the Owner, the Contractor shall have been given notice of all defects or omissions reasonably noticeable before such occupation. No certificate, final or otherwise, shall be construed as relieving the Contractor from his obligations to make good any defects discovered in his work after the completion and acceptance of the same, nor as a waiver of any specific obligation the Contractor may have assumed as to the durability of his works. The acceptance by the Contractor of the payment of the final certificate shall constitute a waiver of all claims against the Owner under or arising

out of the contract. The Contractor, before each payment falls due, shall submit to the Engineer a requisition therefor and shall, if required, submit therewith an itemized statement of the quantities and value of works in general items performed to date, and such statements shall be made in form provided or approved by the Engineer.

The provisions of this section are of great importance in any contract. Its importance is in its wording, and not because it contains any new principle.

XXXI.—Liens.—If at any time there shall be evidence of any lien or claim, for which, if established, the Owner or the premises might be made liable, and which would be chargeable to the Contractor, the Owner shall have the right to retain out of any payment then due, or thereafter to become due, an amount sufficient to indemnify himself against such lien or claim until the same shall be effectually discharged, or bonded; and should there prove to be any such claim after all payments are made, the Contractor shall refund to the Owner all moneys that the latter may be compelled to pay in discharging any lien upon said premises made obligatory in consequence of the Contractor's default. Before the final payment shall become due, the Contractor shall deliver to the Owner a satisfactory release of all liens against the premises on the part of all persons who have delivered materials for use in, or performed work on, said premises, including a release from the Contractor himself.

XXXII.—Unfixed Materials.—When the Contractor shall have received payment of any certificate in which the Engineer shall have stated that he has taken into account the value of any unfixed materials intended for the works, and placed by the Contractor thereon, or upon ground adjacent thereto, all such materials shall become the property of the Owner, and shall not be taken away, except for the purpose of being used on the building, without the written authority of the Engineer; and the Contractor shall be liable for any loss of or damage to such materials.

XXXIII.—Termination of Contract by Contractor.—Should the Owner not pay the Contractor any sum certified by the Engineer within the times respectively named in the Agreement, the Contractor shall give to the Owner written notice of the non-payment, and should the Owner not pay any such sum within the period of .... days from the date of delivery of such notice at the Owner's address, or sent to him there in the ordinary course of mail by registered letter, or if the Owner shall become bankrupt or file any petition for liquidation of his affairs, and if his Receiver in Bankruptcy shall repudiate this contract, or if the Receiver shall be unable to show, within .... days, to the reasonable satisfaction of the Contractor, his ability to carry out

the contract, and to make all payments due or to become due thereunder, or if the works be stopped for .... months, under an order of the Engineer or any Court of Law, the Contractor shall be at liberty to terminate the contract by notice in writing to the Engineer, and to recover from the Owner payment for all work executed and for any loss he may sustain upon any plant or material supplied or purchased for the purpose of the contract. In ascertaining the amount of such payment, the rates contained in the Contractor's original estimate shall be followed, or, where the same may not apply, rates proportionate to the prices therein contained, as far as the same are applicable.

Few American contracts contain any provision permitting a Contractor to terminate a contract on any condition whatsoever. In fact, most contracts are filled with clauses designed to bind a Contractor hand and foot so that, no matter what may happen, he is certain to remain a party to the contract. The absolute unfairness of many such requirements is hard to account for, except that they have been inherited from times when work and conditions were totally different from those existing to-day. The fairness of Engineers including within contracts a section similar to the above should be apparent from a careful reading of the section.

XXXIV.—Arbitration.—Provided always that in case any dispute or difference shall arise between the Owner, or the Engineer on his behalf, and the Contractor, either during the progress of the works or after the termination, abandonment, or breach of the contract, as to the construction of the contract or as to any matter or thing arising thereunder (except as to matters left to the sole discretion of the Engineer under Sections II, VII, X, XIV, and the exercise by him, under Section XIII, of the right to have any work opened up), or as to the withholding by the Engineer of any certificate to which the Contractor may claim to be entitled, then either party shall forthwith give to the other notice of such dispute or difference, and such dispute or difference shall be and is hereby referred to the arbitration and final decision of three disinterested arbitrators, one chosen by the Owner, one by the Contractor, and the third chosen by these two arbitrators. The decision of any two of these arbitrators shall be final and binding on both parties hereto. If the parties hereto mutually agree, one arbitrator may be appointed in place of the three named as above, and his decision shall be final and binding on both parties.

When the time for submitting matters to arbitration, as herein provided, has arrived, either the Owner or the Contractor may appoint an arbitrator and demand the appointment of an arbitrator by the other party. Should the party filing notice of appeal to arbitration as

herein provided fail to choose an arbitrator or to agree on a single arbitrator within seven days after receiving a written request from the other party, his right to arbitration in such matters shall lapse; should the other party fail to choose an arbitrator or to agree on a single arbitrator within seven days after receiving a written request so to do, then such dispute or difference shall be and is hereby referred to the arbitration and final decision of ....., or in the event of his death, or unwillingness or inability to act, of ....., or in the event of his death, or unwillingness or inability to act, of a person to be appointed on the request of either party after giving notice to the other party, by the President for the time being of the American Institute of Architects or of the American Society of Civil Engineers, and the award of such arbitrator shall be final and binding on both parties.

Such reference, except on the question of certificate, shall not be opened up until after the completion or alleged completion of the works, unless with the written consent of the Owner or Engineer and the Contractor. The arbitrator, or arbitrators, shall have power to open up, review, and revise any certificate, opinion, decision, requisition, or notice, save in regard to the said matters expressly excepted above, and to determine all matters in dispute which shall be submitted to him or them, and of which notice shall have been given as aforesaid, in the same manner as if no such certificate, opinion, decision, requisition, or notice had been given. Should either party refuse, or neglect, to supply the arbitrator or arbitrators with any papers or information considered necessary by him or them to arrive at a decision in the matters referred to him or them, which papers or information have been specifically demanded in writing, he or they are hereby authorized and empowered by both parties to the contract to proceed in *ex parte* proceedings, and such proceedings shall be final and binding.

Upon every or any such reference, the costs of and incidental to the reference and award, respectively, shall be in the discretion of the arbitrator or arbitrators, who may determine the amount thereof, or direct the same to be taxed as between attorney and client, or as between party and party, and shall direct by whom and to whom and in what manner the same shall be borne and paid.

This section caused more debate among the members of the Royal Institute of British Architects and more differences of opinion between the British Architects and Builders than all the other sections combined. The discussion on this subject, in the *Journal* of the Royal Institute of British Architects, is especially worthy of study.

The importance of this section justifies the most careful study which can be given to it. Around it would center the much discussed

authority of the Engineer or Architect to settle all matters of every kind and description. Everyone who has exercised such powers must realize that while they are theoretically desirable, their proper curtailment would unravel a very difficult situation, and would permit questions being considered in a more equitable light for all parties.

The usual American arbitration clause is a delusion and a snare to all parties. Many have felt that it was a guaranty of redress for possible wrongs, only to be awakened when its provisions were called into action.

It should be noted, of course, that the English "Arbitration Act, 1889," is a very practical document, and probably gives more legal security to the enforcement of arbitration clauses than would be possible under the laws of most of the States. Arbitration clauses are used in many contracts, and they should be made as practical and workable as possible; then, if it is found that even under the most practical of clauses the State laws need strengthening to make the term "arbitration" a reality and not a myth, there would be a good foundation on which to advocate an arbitration act similar to the English statute.

The above section leaves the Engineer as the deciding party between both Owner and Contractor, but should either party formally dissent from his decision, a notice can be given of an appeal to arbitration. Such appeals, however, will not be actually submitted to the arbitrators until the substantial completion of the work. This seemed to be the point of greatest disagreement between the British Architects and Builders. The former indorsed the above procedure, while the Builders desired the matters to be submitted to arbitrators at any time. The objection to permitting submissions at any time is that, under the broad powers which it is desirable to grant to arbitrators, it would cause confusion during construction, and make it possible to confuse hopelessly the power of the Engineer and the arbitrator. After the substantial completion of the work, such conflict cannot well arise. The Engineer being supreme during construction and after completion, the arbitrator has power to review only certain of the Engineer's decisions to which written exception has been taken, as herein provided. Of course, in the matters which are referred to arbitration, the decision of such arbitration is final. Such a policy, by its very broadness, would tend to make Owners, Engineers, and Contractors more willing

to listen and consider the grievances, real or imaginary, of the other parties.

Section XXXV is intended to dissuade both parties from taking unnecessary or trivial appeals from an Engineer's decision.

While the foundation of the arbitration section is laid on the lines of that of the Royal Institute of British Architects, it differs in many details, and its importance justifies giving the R. I. B. A. section in full, for purposes of comparison.

ARBITRATION SECTION OF ROYAL INSTITUTE OF BRITISH ARCHITECTS.

"32.- Provided always that in case any dispute or difference shall arise between the Employer or the Architect on his behalf and the Contractor, either during the progress of the works or after the determination, abandonment, or breach of the Contract, as to the construction of the Contract or as to any matter or thing arising thereunder (except as to the matters left to the sole discretion of the Architect under Clauses 4, 9, 16, and 19, and the exercise by him under Clause 18 of the right to have any work opened up), or as to the withholding by the Architect of any certificate to which the Contractor may claim to be entitled, then either party shall forthwith give to the other notice of such dispute or difference, and such dispute or difference shall be and is hereby referred to the arbitration and final decision of ....., or, in the event of his death or unwillingness or inability to act, of ....., or, in the event of his death or unwillingness or inability to act, of a person to be appointed on the request of either party by the President for the time being of the Royal Institute of British Architects, and the award of such Arbitrator shall be final and binding on the parties. Such reference, except on the question of certificate, shall not be opened until after the completion or alleged completion of the works, unless with the written consent of the Employer or Architect and the Contractor. The Arbitrator shall have power to open up, review, and revise any certificate, opinion, decision, requisition, or notice, save in regard to the said matters expressly excepted above, and to determine all matters in dispute which shall be submitted to him, and of which notice shall have been given as aforesaid, in the same manner as if no such certificate, opinion, decision, requisition, or notice had been given. Upon every or any such reference the costs of and incidental to the reference and award, respectively, shall be in the discretion of the Arbitrator, who may determine the amount thereof, or direct the same to be taxed as between solicitor and client or as between party and party, and shall direct by whom and to whom and in what manner the same shall be borne and paid. This submission shall be

deemed to be a submission to arbitration within the meaning of the Arbitration Act, 1889."

XXXV.—Damages for Submission to Arbitration.—In submitting to arbitration questions in dispute, or differences involving a variation in the amount of the contract, both the Owner and the Contractor shall have the option of submitting their appeals in separate items or groups of items. In which case the arbitrators or arbitrator, in awarding a judgment on each item or group of items, are authorized and directed by both parties to the contract, to award to the party whose contention is sustained, in addition to the sum actually determined as correct, an amount equal to .....% of such amount. Such additional sum shall be added to or deducted from the amounts in question as the case may require. Should a claim be set aside in whole, or should the nature of the case otherwise require it, the amount of such additional award shall be added to or deducted from the amount due the Contractor on the contract. These additional sums shall be considered as damages for the time and trouble incident to such appeal.

The object of this section is to act as a check to prevent appeals to arbitration becoming the fashion rather than the exception. While the principle of this section is considered good, the ability to enforce it is not settled in the writer's mind.

General Conditions.

The endeavor has been to make the Agreement and Schedule of Conditions of Contract as broad and general as is consistent with good practice. There are additional conditions which may be desirable or essential for particular buildings, but as these conditions are special and of only limited, if not personal, application, it is unwise to attempt to write such in a general set of Conditions of Contract.

Legally, such conditions would be just as binding upon a Contractor whether they were given as a part of the Agreement or were placed in the Specifications. The writer, therefore, advocates placing in the fore part of a Specification a chapter headed "General Conditions," under which would be placed special requirements of a more or less legal and general nature, which, for the purpose of the particular building, might have been made a part of the Schedule of Conditions of Contract.

The following are given as samples of clauses which might be found under a heading "General Conditions." It must be distinctly

understood that these clauses are given only as samples, without comment. The writer does not endorse them for general use, and, except in isolated cases, he considers that many of the provisions might be more harmful than beneficial.

Checking Documents.—The Contractor shall carefully study and compare all drawings, specifications, and other information given to him by the Engineer, both as to figures, materials, and methods of construction, and shall report in writing to the Engineer for rectification any error, inconsistency, or omission therein, which he shall discover, or any modification he considers necessary or desirable for the stability or economical construction of the building.

Doing Work in Freezing Weather.—Work shall not be done when the temperature where work is being done is below 28 degrees Fahr., or if exposed to rains, except by special permission of the Engineer. Frozen materials shall not be built upon.

Engineer's Approval.—All materials or work specified to be of an "approved" type, construction, or finish, shall be submitted to and approved by the Engineer before being used or executed. Where materials, quality, or finish are not distinctly specified, samples shall be submitted to and approved by the Engineer before using same. An approval by the Engineer shall be understood to mean that it is conditional upon the work being in conformity with the contract requirements. No approval of deviation from contract requirements shall be implied, but shall be specifically mentioned in the Engineer's written approval. (See Section IV, Schedule of Conditions of Contract.)

Materials and Workmanship.—All materials shall be new, and all materials and workmanship shall be the best of the respective kinds contemplated by the contract, unless otherwise specified.

Contemplated Work.—The work to be executed under the contract consists in general of a ....., constructed and completely finished in every particular ready for furnishing, within the limits shown on the Drawings and described in the Specifications, except (*a*) elevators, (*b*) lighting fixtures, (*c*) electrical, (*d*) heating work, (*e*) or work specifically excepted or made subject for separate bids.

Contract Documents.—(See Agreement, Article I.)

Definitions.—The following definitions of works, terms, etc., shall be understood to govern their use throughout the Contract:

> *Approved.*—The word "approved," as applied to any material, manner or mode of construction, shall be construed as though followed by the words "by the Engineer."
>
> *Best.*—The term "best," or "best quality," shall be taken to mean that there is no better quality or finish of material of the

grade specified or reasonably implied, obtainable on the market supplying the vicinity, or that there is no better class of work mentioned, reasonably obtainable.

Net Cost.—(See Conditions of Contract, Section XXVII.)

Provide, Etc.—For the sake of brevity, and to avoid needless repetition, the imperative mood has been largely used throughout the specification. Wherever it is used it shall be understood to mean, "The Contractor shall furnish all material and perform all labor necessary to .....," and these words shall be read into the sentence.

Provisional Amount.—(See Conditions of Contract, Section XXVIII.)

Drawings.—The Contractor shall furnish to the Engineer at proper times all shop and setting drawings or diagrams which the Engineer shall deem necessary to carry out the work. Each drawing or diagram shall give all the information required by the nature of work which it is intended to govern. These drawings or diagrams shall conform strictly to the Engineer's drawings and specifications, and where possible upon measurements taken from existing work. No deviation from the Engineer's drawings and specifications shall be made unless by special written order. The Engineer will inspect such drawings, but his purpose being to aid the Contractor and expedite the work, he shall not be held responsible for the Contractor's errors, the result of which, should they occur, must be made good at the Contractor's expense. The Contractor shall make required changes in such drawings or diagrams and supply the Engineer with one correct copy. Any models or templates submitted shall be changed as reasonably required until satisfactory.

Fires, Smoking, Etc.—The Contractor shall not permit open fires to be lighted at any time in or dangerously near the building, and, after the roof is on, shall prohibit smoking within the building.

Imperfect Construction.—The Contractor shall notify the Engineer in writing if at any time he finds that any work or material that has or is being furnished by another Contractor in connection with his work is imperfect; and in case of his failure to give such notice, the Contractor shall be held to have accepted such work or materials, and all other existing conditions, and shall be responsible for any defects in his own work consequent thereon, or any change to his work made necessary by the future correcting of such defective work by other Contractors, and shall not be relieved of the obligation of any warranty because of any such condition or imperfection.

Imperfect Design.—The Contractor shall notify the Engineer in writing if he thinks any part of the work as designed or constructed is unsafe or likely to become so, or unsuited to produce the intended

results, and shall thereafter proceed with such work only on the order from the Engineer.

Lines and Levels. The Contractor shall establish all lines and levels necessary to the proper locating, laying out, and erecting of the work, in accordance with the Owner's survey and Engineer's drawings, and shall be responsible for their accuracy and maintenance. (See Schedule of Conditions of Contract, Section V.)

After the foundations have been completed and before the first tier of beams is set, the Contractor shall file with the Engineer a certificate from an approved surveyor showing that the building is located correctly according to the Owner's survey, and that the foundations, column bases, etc., are correct according to the drawings.

Measuring Work in Place.—The Contractor shall measure work already in place to insure the proper execution of subsequent work, or to assist the Engineer in making his drawings, and should any discrepancy between the executed work and the drawings be discovered, he shall report at once such discrepancy to the Engineer.

Notifying Engineer.—The Contractor shall bring to the attention of the Engineer any conditions pertaining to the site, the soil, the various portions of the work, whether executed by himself or by other Contractors, or to proposed work, or to laws and ordinances governing the work that should be known by the Engineer.

Overloading Structure.—The Contractor shall not permit any part of the structure to be loaded during construction with a weight greater than it is calculated to bear.

Reports to Engineer.—The Contractor shall furnish reports to the Engineer showing the status of the work at the building and in the shops, whenever so required, and in the manner directed.

Signs.—The Contractor shall not permit signs upon the building or premises, other than that of the General Contractor, except by the written permission of the Engineer.

Storage Limits.—The Contractor shall confine the storage of materials and the operations of his workmen to the reasonable limits indicated by the Owner or Engineer, and shall not unnecessarily encumber the premises with his materials.

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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE EFFECT OF ALKALI ON CONCRETE.

By George Gray Anderson, M. Am. Soc. C. E.

To be Presented February 16th, 1910.

To some extent, this paper is a continuation of the informal discussion on "Irrigation" which took place at the Annual Convention of the Society, held in Denver, Colo., in June, 1908.*

In that discussion some reference was made to the introduction of more permanent structures in irrigation work; that, for such, concrete was being used extensively; and that, in some smaller structures, failures had occurred, in the form of crumbling or disintegration, due, presumably, to the action of the alkali frequently present both in the soil and in the water, and resulting from chemical reactions between the sulphates in these waters and the cement, which reactions may not yet be fully understood and are under investigation. Since that time the subject has received more than passing attention from the Profession and the industry at large, as shown by more or less extended comments in the technical and trade journals.

Several instances of such failures were referred to by members who took part in that discussion, and it is largely the purpose of this paper to consider these and other instances somewhat more fully, to elicit further discussion and consideration, and, if possible, to arrive at more

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

**Transactions*, Am. Soc. C. E., Vol. LXII, p. 1.

definite conclusions in reference to the disintegrating action of alkali, in the soil and in the water, and of the methods of counteracting it.

It should be stated, at the very outset, that the subject is not approached with alarm, but rather with an inquiring mind.

Irrigation development has passed the stage in which temporary structures are considered; and the Engineer is looking for permanence, from the economic standpoint of reduction in cost of maintenance, operation, and renewals. He is almost ready to consider favorably the use of concrete, in all forms and for nearly all structures. Under observation, certain defects, arising from natural conditions which are almost always present in the regions of operation, have become apparent; and these cause a temporary hesitation and further consideration of plans and methods. The attitude is by no means that of abandonment, but rather of painstaking investigation of all the conditions, with the desire to ascertain the cause, and, from that, to introduce methods which will overcome the defect. Most certainly it is not the desire to magnify unduly the significance of a few isolated instances of failure, or to ignore defective constructional methods in these, but rather to investigate methodically all phases of the failed structures. Such investigation is now being made by the chemical branch of the Reclamation Service, as well as by others, and until official reports are published, such discussions as the present may be considered premature. In view, however, of the great amount of work under way and in contemplation in regions known to be affected, and the serious importance of these matters, not only to irrigation engineers, but to all members of the Profession engaged in structural work using concrete in contact with alkali (in soil or in water), it is not out of place to add the available knowledge and information to the more extended and methodical inquiries.

It is well known, of course, that alkali is present in greater or less amount in the soil of the arid regions. In the disintegration of the rocks, the minerals have been broken down into mineral salts which have remained in the soil as soluble salts. The rainfall and the drainage of the region are insufficient to carry these salts out of the soil through the subsoils and eventually into the streams and rivers, as is the case in the more humid regions.

The accumulation explains the wonderful fertility of the soil in arid regions, and at the same time accounts for the menace to the purely

agricultural interests. In sandy soils, with free drainage, irrigation assists in the removal of these salts; in the closer soils, such as shale, with less drainage, it may and does increase the accumulation to a harmful degree.

All are familiar with the results in connection with agricultural operations. Efflorescence in large amounts is a sure indication of alkali, and then it is in its least harmful form. What is known as "black alkali" is much more injurious to plant life. The latter form represents the alkaline carbonates, which actually dissolve the organic materials of the soil, and kill the germinating seeds or roots of the plants; the former, the white alkalis (of which the most common are sodium sulphate, sodium chloride, magnesium sulphate, and magnesium chloride), are not in themselves poisonous to plants, nor do they attack the substance of the plant roots; but they are injurious when in excessive amounts, as they prevent the plants from taking up their needed food and water supply.

It has been tolerably well determined just what amount of soluble salts is limiting for plant growth, varying with the character of the salts, of the soils, and of the plant itself; for crops differ in their ability, and some will thrive on soils on which others would languish.

Such, very briefly, is the general action of "alkali" on agricultural interests. The salts are largely the sulphates of lime, magnesia, soda, alumina, iron, and potash, and the carbonates of soda and potash. Of these, the sulphates only may prove injurious to concrete, and as the shales of the Dakota formation are known to be rich in alkalis, and are common throughout the arid regions, the effect of these soils and their salts on concrete structures will be set out by brief reference to a few known cases where this action has taken place.

A good many, if not all, of the projects of the Reclamation Service are located in alkali regions, and, thus far, the attention of the engineers has been directed to the failure of concrete structures on only two of these projects—one in Montana and one in Wyoming.

These failures may indicate some special peculiarity of the local soil, and may be somewhat similar to the well-known fact that the action of sea water has been destructive to concrete in certain localities and not in others.

The Service has started an investigation, which is still continuing, and the chemist's report has not yet been published, but Mr. Jewett,

the cement expert of the Service, in a paper before the American Society for Testing Materials, set forth some of the conditions, as will be shown by the following extracts*:

"On the Sun River project * * * its effect has been seen in the disintegration of a number of small structures, such as pipe culverts, which are partially submerged in the small streams carrying the drainage of the surrounding country. The effect of this water in causing the disintegration of the concrete is noticed, both in the softening of the material where constantly submerged for a period of three or four months, and also more particularly in the breaking up of the structure at the line where the water rises and falls, or, in other words, where there is a submersion followed by a drying out, followed by another submersion, etc. This process brings about a breaking up of the structure, similar in appearance to that of the effect of freezing, only much more severe. It is described by the engineer in charge of this work as follows:

" 'On examination of some of the concrete culverts which have been in place in alkali water for three or four months, the part under water is much softer than the part above water, but the greatest effect of the alkali seems to be at the water surface. Have examined pieces of these culverts taken from below water level and found, by microscopic examination, minute crystals which disintegrate on being exposed to the air and become a fine powder. This process of crystallization has an enormous expansive force, the same as freezing, and this undoubtedly has an enormous disintegrating effect. One culvert which had been in alkali water several months had no strength at all and was like so much sand and mud at the water level.'

* * * * * * *

"In studying the cause of this disintegration it seemed that a chemical analysis of the water producing these results would be the first step of importance to be taken. The results obtained from an analysis of this water are as follows:

	Milligrams per Liter.
Calcium sulphate ($CaSO_4$)	1 690
Magnesium sulphate ($MgSO_4$)	6 870
Basic magnesium carbonate (MgH_2CO_3)	305
Magnesium chloride ($MgCl_2$)	192
Potassium chloride (KCl)	20
Total solids	9 077
Weight after ignition	8 855
Loss on ignition	222

* *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, 1908.

"From this analysis it will be seen that the principal mineral ingredient of this water is magnesium sulphate, and this leads to the conclusion that the effect of this water in causing disintegration of concrete is similar in its action to that of sea water in which magnesium sulphate is stated to be the principal substance acting to bring about the disintegration of concrete with which it comes in contact."

Professor Headden, of the Colorado State Agricultural College, has at various times issued bulletins in reference to alkali in the soil and in the water, and has recently added to his publications Bulletin 132 on "The Destruction of Concrete by Alkali," in the course of which he gives an account of his investigation of cement tiling which had rapidly deteriorated. From that bulletin the following is quoted in order to convey clearly the conditions.

"The following case of deterioration may be of some interest as it presents some new points. There were sent to me some months ago a number of fragments of tiling which had been laid but from eight to nine months. They had been disintegrated to such an extent that one of the samples was simply a white putty-like mass mixed with sand. There was nothing about this sample remotely suggestive of concrete. Another of the samples consisted of a fragment of the tile, the interior portions of which had been wholly decomposed, while there still remained an outer and inner portion in good condition, or comparatively so. Another sample had been attacked in the inner side leaving the mass of the tile outside of the zone of decomposition apparently sound. The line of decomposition was sharply limited and showed distinctly, but even in this case there remained a thin layer of cement on the inside.

"I do not know whether the tile were running full of water or not, probably not, as the fragments seem to be parts of 16 or 18-inch tile. The importance of this point is simply this, that it would answer any question in regard to the separation of solid 'alkali' salts on that portion of the tile above the water line. That such a separation of these salts should take place does not seem very probable in this case.

"The point of attack, so far as the samples at my disposal indicate, is either in the center of the cement mass or near the inner surface. In either case we are almost compelled to assume the action of the water, *i.e.*, of the salts held in solution. At the same time we see the resisting power of the outside and inside surfaces, which are evidently richer in cement than the inner portion of the mass of the tile.

"The sand used in making the tile was quite fine. The proportion used was 5:1. We can readily see that the individual masses of cement in the interior mass of the tile must be small and readily attacked by solutions of salts capable of reacting with the cement.

"It seems quite evident that the agents causing this disintegration of the cement must gain access to the interior portion of the mass of the tile in the form of a solution, but it is a question whether this solution is the same in strength and character as the water in the drain. I think that I have clearly shown in Bulletin 65 of the Colorado Experiment Station, page 34, that the water soluble portion of the soil is different from the salts held in solution in the ground water. Further, in Bulletin 72, page 25 *et seq.*, that the ground waters are different from the drain waters, and the action of these waters on the tile would vary somewhat according to the source of the solution which penetrated the mass of the tile.

"The drain waters obtained, say at a depth of four feet, contain as a rule a smaller amount of salts in solution than the ground water. The salts present are the same, but their relative quantities vary. The usual salts present in the ground waters are, sodic sulfate, calcic sulfate, magnesic sulfate, sodic carbonate, and sodic chlorid, in some instances also magnesic chlorid. In the drain waters we have the same salts, but their relative quantities are different, the most notable feature being the relatively large amount of sodic carbonate.

"The substances claimed to be the most active in effecting the disintegration of cement are the sulfates. In the case of sea water the explanation offered is that the lime of the cement decomposes the magnesic salts present in the water, whereby the lime goes into solution and the magnesia is deposited in its stead, causing a bulging and disintegration of the work owing to its greater bulk. The principal magnesian salt in sea water is the chlorid, the deleterious action of which on cement is not established, but it has been shown that magnesic sulfate acts energetically. The most active agent in decomposing concrete is the sulfuric acid of the sulfates carried in solution. Again, solutions of gypsum have been shown to act detrimentally on concrete by forming with the tri-calcic aluminate a sulfo-aluminate.

"The water acting on these tiles, whether it be ground water or drain water, is very bad. One of the two small samples sent to me contained 1 252.6 grains per imperial gallon, of which 493.5 grains were sulfuric acid (SO_3) with an extremely large proportion of magnesia. The other contained 542.3 grains per imperial gallon, with 42.31 grains of magnesic oxid or 7.8 per cent. of the total solids. I have examined a number of waters from this section, and find the ground water uniformly heavily laden with the sulfates of lime and soda, with comparatively large quantities of magnesia, especially if the water comes from near the surface, as it necessarily must in the case of drain waters. In addition to the above constituents, there is uniformly some carbonate present, which for the present purpose we may consider as sodic carbonate.

"We have then an ample supply of these substances present which

are known to act deleteriously upon cement, *i. e.*, the sulfates, a portion of which is magnesic sulfate, and in addition there is so much sodic carbonate present that its action cannot be neglected."

Professor Headden, assuming that the cement used in the tiling was an average Portland cement, with not more than 2% of calcic sulphate, then considers the action of the waters on the tile.

The white decomposition product indicated the presence of a large quantity of water, also of considerable quantities of sulphuric and carbonic acids.

The decomposed mass was washed with distilled water, and the wash-water showed that it had taken sulphuric acid, lime, and soda into solution.

An analysis of the Arkansas River water at Rocky Ford, which, Professor Headden remarks, is essentially a return water, is given, and the assumption is made that the composition of the water acting on the decomposed tiling was much the same, as are all waters in that district, varying in amounts of mineral matter, or total solids held in solution, as shown by the two samples of drain water accompanying the tiling—one having 1 252.6 grains, the other 542.3 grains.

The action on the tiling seems to be similar in Colorado and Montana. A comparison of the features of construction may not be possible.

In the Montana case, Mr. Jewett says:

"Of course, in this case, as in any other case of concrete failure, the question of poor workmanship is one of the first to be considered and it should be noted that the work in question was carried on by force account under direct supervision of employees of the Service and not by contract, and was done in the usual thorough and substantial manner, in which all of the construction work of the Service is carried on. After taking all the evidence into account there seemed to be no other conclusion but that the action of the alkali water was directly responsible for the results noticed."

Professor Headden has not much information on the quality of the tiling examined by him. The sand used was quite fine, the proportion 5 to 1, and the individual masses of cement in the interior must have been small and readily attacked by solutions of salts capable of reacting with the cement.

Both Mr. Jewett and Professor Headden find the decomposing action, in the cases of these tilings, to be similar to that of sea water,

in which magnesic sulphate is stated to be the principal agent. Professor Headden says, as already quoted,

"The principal magnesian salt in sea water is the chlorid, the deleterious action of which on cement is not established, but it has been shown that magnesic sulfate acts energetically."

Reference to this similarity to the action of sea water will be made later; meantime, the following is a summary of Professor Headden's conclusions.

The analysis may be interpreted as indicating that the cement had been deprived of silicic acid and of a relatively large amount of lime. The amounts of the ferric and aluminic oxides have been reduced to about one-half as much in the decomposed cement as in the original. The sodic sulphate, more particularly the calcic and magnesic sulphates which are present in the ground-water, will at the same time act on the tri-calcic aluminate, forming the sulpho-aluminate, which crystallizes with 60 molecules of water and is soluble. The increase of sulphuric acid (more than 20%), and also of carbonic acid (more than 12%), in the water-free decomposition product is noted, as compared with only about 1% each in the original product, while magnesic sulphate has also contributed to the result, being more than four times greater in the decomposed product than in the original; and there is the conclusion that:

"The action of the ground waters seems to have been one of chemical change, converting the lime of the cement into a sulfate and carbonate accompanied by the removal of silicic acid, alumina, and lime, and totally destroying the cohesiveness of the concrete."

The case of the sewers at Great Falls, Mont., has been referred to frequently, and is probably the most striking illustration of deterioration yet considered. The City Engineer, Mr. C. W. Swearingen, was so impressed by the conditions that he requested the authorities at the Montana State Agricultural College to investigate and report on it.

The preliminary conclusions of Mr. E. T. Tannatt, Rural Engineer, and Mr. Edmund Burke, Chemist, are embodied in Bulletin No. 69 of the Montana Experiment Station, from which the following extracts give a very clear exposition of the conditions:

"The Third Street main sewer was constructed in the year 1890, the material used being Portland cement pipe of home manufacture, made in moulds and put in place after the cement had hardened.

Sewer is oval in shape, dimensions 26 x 32 inches. This sewer, after a comparatively short term of service, showed defects of so serious a character as to necessitate rebuilding portions of the same. The remaining portion of this sewer has disintegrated in many places and occasionally portions of the pipe are gone, the cement in places has the appearance and consistency of fresh lime mortar.

"The Sixth Street main was constructed in the year 1892, the material being the best grade of common brick of home manufacture. Sewer is circular in form, four feet inside diameter, two rings of brick laid on edge in imported Portland cement mortar, outside plaster coat one-half inch thick, not plastered on inside; all cement mortar one part cement and three parts sand. The masonry in the invert in this sewer is disintegrating throughout its entire length, the defects being confined almost entirely to the invert and lower sections of the manholes; the arch is in apparent good condition.

"There is no sub-drain under, nor on the sides of the sewer. As stated before, the disintegration in the Third Street main, where cement pipe was used, commenced soon after its construction, whereas the Sixth Street main gave no evidences of defect for about seven years after its construction, and it is only during the past four years that decomposition has been rapid."

The examiners found: (*a*) that the disintegration and destructive action of the cement did not come from adulteration, nor from the quality of the materials used; (*b*) that though a limited chemical or physical action of the alkali salts may be a partial cause of the breaking down of the brick, yet the principal cause was the fact that the bricks were too soft, and not fit for sewer construction; and (*c*) that the action on the cement was chemical, that the destructive agent came from the outside of the sewer, and that the disintegration of the cement cannot be charged to any action of the sewage or gases from the same.

They found that certain portions of the sewers showed the presence of external moisture to a more marked degree than other sections, and that, contiguous to it, were various depressions in which water collected and gradually seeped into the soil, thereby affording means for the leaching out of the alkali from the gumbo, through the sand and other more permeable formations, into the back-filling adjoining the sewer, there depositing against the structure. The soil samples showing the highest percentages of alkali were taken immediately at the back of the cylinder of the sewer.

Soil samples were analyzed in order to ascertain the amount of alkali salts, whether these were being leached out and lodged against

the sewer, and the amounts at various depths, but there is no chemical investigation of the action on the cement and its alteration, which it is understood will form the subject of a later bulletin.

The analyses embodied in the report, however, show that the maximum disintegration of the sewer occurred where the greatest percentage of alkali showed in the soil, and that the ground-waters were leaching the alkali from the soils and depositing the salts along the sewer.

Messrs. Tannatt and Burke consider that the use of cement for sewers and other structures in alkali soil is inadvisable in cases where it is not possible to drain all ground-waters from the exterior of the sewer, and they recommend that drain tile be placed in the bottom of the trench, covered with clean gravel or crushed stone, and that similar drains be placed around the barrel of the sewer to provide effective drainage of the ground-waters, in order that it will not be possible for them to come into contact with the structure itself, and, where these precautions cannot be taken, they "unhesitatingly recommend that cement pipe be not used."

The writer had some experience with a construction somewhat similar to the Great Falls sewer, and in similar, though not such aggravated, conditions, but with entirely different results.

Lake Loveland was constructed under the writer's supervision in 1894. The outlet is a tunnel about ¾ mile long, nearly all of which is in stratified shale. The rock was of such a character that it would have decomposed under the action of passing waters at a high velocity, and therefore the tunnel was lined. The interior diameter of the tube was 5 ft. The invert was built of two rings of well-burned brick, and the arch was of concrete.

In the excavation, large volumes of water were developed, ultimately giving a flow of about 2 cu. ft. per sec. The water, undoubtedly, was impregnated heavily with alkali, but the writer did not obtain a chemical analysis of it, as, at that time, he did not suspect that any deleterious action might be caused by the character of the water itself; but, apart from the fact that it was "return water"—the drainage of a large section of irrigated country lying west of the reservoir—there were constant physical evidences that it contained salts, if one might judge from the action on the human system caused by drinking it in moderate quantities.

Through the whole length of the tunnel a sub-drain was channeled out in the rock bottom, and wooden boxes were placed to convey these waters. Side-drains were also placed at frequent intervals throughout the whole length, and mostly where the volume was largest, always connected with the under-drains. These were carried up as high as the drainage showed, usually to the spring of the arch. In short, every precaution was taken to prevent the water from coming in contact with the concrete, certainly until it had set thoroughly, and solely for the purpose of securing good results and without thought of any ultimate chemical reaction.

The work was done practically by force account. The materials were supplied, the labor was done under contract, and the work was in charge of a competent concrete worker and under a thoroughly able inspector.

As far as the writer knows, the concrete and the cement mortar in the brickwork are in thoroughly good condition to-day. It is nearly 8 years since he examined the work, and then it was found to be without any apparent defect. The superintendent informed the writer, recently, that he had not been in the tunnel for 2 years, but had then found it in perfect condition, and was of the opinion that it still is so.

One other structure may be mentioned, a reservoir spillway in the Arkansas Valley, or the Bob Creek Reservoir, constructed and owned by Mr. W. C. Bradbury. The writer's attention was attracted to it in September, 1909, when passing near it, on other business. He first noticed and examined a small division head-gate about ½ mile below the reservoir. It was stated, that these works had been built less than 2 years before that time. The writer had no means of knowing the proportions used in the work, but this head-gate seemed to have been fairly well built, and its upper portion was in fair condition. The floor had evidently been about 6 in. in thickness, but it was entirely decomposed, and could be crushed readily in the hand. The uprights, up to about the normal height of the water, showed disintegration. At the outlet from the reservoir, the apron was disintegrated in the same way. It had been reinforced, and the reinforcing material was exposed and bright while the concrete mass was almost completely pulverized.

The spillway to the reservoir was of considerable length and height, of ogee overfall, and had the same appearance of fairly good construction. Prior to the writer's visit, there had never been any water flow-

ing over it from the reservoir. Its lower portion, for a height of about 10 ft. above the apron level, was markedly discolored, and on breaking the surface crust, it was found that the interior mass of the concrete could easily be removed and pulverized in the hand.

About a month later, water passed over the spillway and washed out the lower section of the fall to the height already indicated. The concrete had been placed on a foundation of shale, from which the alkali water oozed. No provision had been made for the under-drainage of this water, and the conclusion is inevitable, that the destruction of the concrete was the result of the chemical action of the alkali in the water draining from the shale.

From these various instances, it would seem that somewhat new problems are presented to the constructing engineer, or, if they are not new, they are old problems under new conditions, and engineers have been prone to overlook their significance until serious consequences have forced them directly on their attention.

It is very likely true that the effects of alkali on concrete are similar to those of sea water, with some differences, as these in themselves differ in various localities. Means of overcoming the sea-water action are fairly well outlined, though as competent an authority as Feret says:

"No cement or other hydraulic product has yet been found which presents absolute security against the decomposing action of sea water."

Michaelis, on the other hand, long ago made certain recommendations which may be summarized briefly here: From the physical point of view, a dense concrete surrounded with an impermeable layer composed of 1 part of cement and 2 or 2½ parts of sand of mixed grains, of which at least one-third must be very fine, and the whole mass made, if possible, in one operation; from the chemical point of view, the use of cement rich in silica and as poor as possible in alumina and ferric oxide, and the use of substances which render the mortar, at any rate in its external layers, denser and more capable of resistance.

The solution of the problem, from the chemical point of view, will be the care of the chemists and the cement manufacturers. The problem for them is as new—or as old under new conditions—as it is for the engineer; and, conceding the general proposition that the solution lies in good construction, which the engineer must realize and will provide, there is, it would seem, the larger problem of a coating which,

adding impermeability, shall also resist the chemical reactions between the soil and water and the cement.

What is good construction under these conditions?

It seems to the writer that the engineer, under these conditions, first has to extend materially the field of his investigations before outlining plans. The character of the soil, and of the water passing over the soil and to be transported over the structure, must be thoroughly analyzed. Then, to the physical problem, similar analytic consideration must be taken of the materials used as concrete aggregates—the sand, the gravel, and the water.

On that feature, the following is quoted from Mr. Jewett's paper:

"In the case of the sand used on the Reclamation Service project above mentioned, the engineers reported that there were evidences that this sand, which was the only supply readily available, contained alkali or some substance making it chemically active and assisting in the disintegration of the concrete. A chemical analysis of this sand was therefore obtained and resulted as follows:

	Per Cent.
"Silica ($Si\ O_2$)	67.03
Alumina ($Al_2\ O_3$)	6.85
Ferric oxide ($Fe_2\ O_3$)	2.50
Manganese oxide ($Mn\ O$)	.02
Lime ($Ca\ O$)	10.72
Magnesia ($Mg\ O$)	1.72
Sulphuric anhydride (SO_3)	Trace.
Alkalies { Soda ($Na_2\ O$)	1.74
Alkalies { Potassa ($K_2\ O$)	1.51
Water at 100° C.	.14
Ignition loss	8.16
Total	100.39

"This sand gives excellent results from physical tests but since, as will be seen from the above analysis, the alkali content is about 3 per cent. which would correspond to about 15 to 20 per cent. of feldspathic material and that it also contains a considerable percentage of limestone, it is altogether possible that this sand would be more readily acted on by the influences tending to disintegrate the concrete than a sand of a more highly silicious nature."

It is extremely likely that similar conditions will be found in most of the sands and gravels available in the arid regions where alkali is known to exist.

There is no occasion for elaboration on the care required in the selection of water for use in mixing concrete. It is quite well known that the ground- and drain-waters in most of these districts are largely impregnated with alkali; that is discernible without chemical analysis, and is particularly true in the lower reaches of irrigated valleys, where most of the waters are drain-waters, consisting of the concentrated essence of alkali from the irrigation and drainage of the upper valleys. Prior to the introduction of the gravity system at Greeley, for instance, the domestic supply from underground galleries contained as much as 1 800 parts of hardness, and, it may be added, the water used in mixing the concrete at the Loveland work referred to, was obtained, not from the drainage of the tunnel, but from the town supply withdrawn from the river inside the foot-hills.

If the action of the alkali salts in the water surrounding a concrete mass can have the destructive effect shown in the various instances quoted, it is certain that the incorporation of such water in the aggregate cannot be other than injurious.

Beyond extra care in the materials, and the production of a dense mass, there is no room to doubt the advisability of effective drainage around the structure.

Professor Headden, in Bulletin 72,* speaking of the alkali lands in the Poudre Valley, said:

"I am convinced that the only question of alkali that we have resolves itself into one of drainage, and beyond this, there is no alkali question for us."

It would be too much to hope that the drainage around structures would as effectively solve the difficulty of alkali in contact with concrete, but it will aid in resisting the action of the agencies at operation.

It is not possible, however, in all cases to provide such drainage as would be effective, and beyond all that—beyond the external application and the possible incorporation of an impervious coating in the mass itself—it seems to the writer that there must be added to it or in it, in some form, agencies which will counteract the effect of these apparently well recognized chemical elements which bring destruction: and that must be left to the chemist, on whose province the writer has probably encroached too far, and, apparently, with only a speaking acquaintance with that science.

* Montana Agricultural Experiment Station

It has been the writer's purpose to bring the subject to the attention of the members, with, perhaps, the selfish motive of eliciting discussion and enlightenment on a subject on which he is looking for light and guidance, in a continued consideration of what seems to be one of the most important subjects than can secure the attention of Western engineers, at this time. The writer is aware that there are others who are far more competent to treat this subject learnedly and practically, and will be content if his efforts have been sufficient to draw from them the knowledge and enlightenment which they possess.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY FOR THE LOCK CANAL AT PANAMA.

Discussion.*

BY MESSRS. C. E. GRUNSKY, H. F. HODGES, THEODORE PASCHKE, AND ALLEN HAZEN.

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E.—In an article on the type of the Panama Canal,† prepared by the speaker in January, 1909, attention was called to the water requirements of the Canal, as estimated by the Board of Consulting Engineers of 1905, on the basis of figures furnished by General Henry L. Abbot, of the Corps of Engineers, U. S. Army (Retired). At that time it was stated that these figures required revision because, after the estimates had been made, the actual area of Gatun Lake was found to be 164 sq. miles instead of 110, as assumed, and because, in the meantime, it had been determined to widen the canal locks to 110 ft. Further discussion of this important feature of the Canal problem by the Canal authorities has not come to the speaker's notice.

Mr. Sorzano's paper is timely because, although it is now too late for a material change of canal plan, it would be unfortunate if proper measures were not taken to bring the lock canal to the highest state of efficiency.

The speaker is among those who were disappointed when the report of the Board of Consulting Engineers—eight to five—was set aside, and the lock canal was adopted. Locks are not added to canals to improve them. They are, however, oftentimes necessary in order to adapt a canal to the physical peculiarities of the available canal route. Where

*This discussion (of the paper by Julio F. Sorzano, M. Am. Soc. C. E., printed in *Proceedings* for October, 1909, and presented at the meeting of November 17th, 1909, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Popular Science Monthly*, May, 1909.

Mr. Grunsky. there is a choice between an adequate canal without locks and one with locks, the question to be answered is whether the better canal is worth the additional cost in time and money. In the case of the Panama Canal this question has never been answered conclusively. The Board of Engineers of 1905, with the data then available, told President Roosevelt that the additional cost of a sea-level canal was justified. The Canal Commissioners, who, in the light of more complete data, might have taken up the question again as late as 1908, seem to have been content to place before Congress and the American people evidence that the Gatun Dam can be made a safe structure, but they failed to furnish convincing evidence that the better canal would not have been worth the additional cost. Moreover, in the light of the favorable progress reports relating to the excavation of the canal prism, it is reasonable to believe that the excess cost would not have been as great proportionately as was originally assumed.

When the American engineer is asked why the United States did not build a first-class canal, he will be at a loss for a satisfactory answer. Nor is it likely that conclusive evidence on this point will ever be furnished by anyone. The time has passed when it was essential to study the larger problem, and, in any discussion, under comparison with the accomplished results, the sea-level canal, with difficulties of conjectural magnitude, would be likely to fare badly.

It seems to be taken for granted that a dam at Alhajuela would safeguard the water supply. The speaker has been at the Alhajuela dam site, and was impressed by the lack of adequate information relating to the geological formations at that point. Exhaustive studies may have been made since that time. At any rate, this word of caution against accepting the Alhajuela site as the best, without full investigation, will not be amiss. At Gamboa, where it would have seemed natural and reasonable to construct a dam, with spoil from Culebra Cut, to an elevation of about 200 ft. above sea level, there is good conglomerate foundation, the lowest point of which is about at sea level. This was satisfactorily demonstrated by borings in 1904. It may still be possible to construct a dam at this point. The Gamboa Reservoir, with a water surface of more than 30 sq. miles, would be a most excellent regulator of the flow of the Chagres River. If this dam had been constructed at once (1905-1907), it would have solved numerous water-supply problems, and would have furnished power for many uses along the line of the Canal. Perhaps a dam can be constructed at this point after the lock canal is in service, but this does not seem likely, because the dam site will be submerged by the waters of Gatun Lake to a depth of about 40 ft. If the work already done has not made this dam impracticable—construction with spoil from Culebra Cut being assumed—the advisability of constructing it at once should receive more than passing consideration by the Canal authorities. If, upon a revision of

the author's figures, it be found that a shortage of water is to be feared, a decision should at once be reached with regard to the location of the additional storage reservoir. This decision should not be postponed until what, at present, seems to be the best site has been rendered impracticable. Mr. Grunsky.

As one of the engineer members of the Isthmian Canal Commission of 1904-1905, the speaker had hoped to have an opportunity of studying and passing upon all the larger engineering problems which the construction of the Canal involves, but this hope was not realized, and since his brief connection with the work, time and opportunity for a comprehensive study of these problems could not be found. Lacking complete information, yet being frequently credited with its possession, it is always with some misgivings that he participates in the discussion of the Canal problems. He wishes to avow himself deeply interested in the success of the Canal, though venturing some criticism.

The water-supply estimates presented by the author are demonstrable in part only. The quantity of water required for lockages, the leakage at structures, the water required for power, the rainfall, and the evaporation from the water surface of the lake can be estimated with some degree of precision from known facts; but the loss of water from wet marginal lands and the loss by infiltration from a lake of large extent are matters of conjecture, in which there is wide latitude for personal opinion. Would it not be better, therefore, to estimate the available supply—the known flow of the streams, the water available from storage, and the rain on the lake's surface—and to deduct from this the losses by evaporation from the lake, leakage at the structures, and a definite allowance for power, and see what remains? The remainder will be the water available for lockages and for the indeterminate losses by marginal evaporation, by seepage, or otherwise. These losses may turn out to be small. Perhaps it will be safe to assume no loss by evaporation from marginal wet areas, because this loss is offset by the gain resulting to the flow of the streams by the elimination of the swamps and wet areas at present within the lake contour above Gatun, for which the author has made no allowance. The loss of water by infiltration and all other indeterminate losses can then be introduced into the calculation as personal judgment may dictate.

The Canal capacity, as determined by the water supply, on the extreme assumption that such indeterminate loss is too small to be considered, cannot be exceeded except by drawing more water from storage. A draft of more than 5 ft. on Gatun Lake, as pointed out by Mr. Sorzano, would reduce the effective depth of the Canal, unless storage be provided elsewhere than in Gatun Lake.

Using for illustration such data—not verified—as are at hand, it would appear that:

Mr. Grunsky.

The low-water flow of streams entering Gatun Lake during four dry months will average about	1 200	sec-ft.
The water obtainable from storage in Gatun Lake, 5 ft. (87 to 82 ft.), for the same period, will amount to	2 210	"
The rain on the lake's surface, about 2.60 in. during four dry months, will be	100	"
Total	3 510	sec-ft.

There will be required:

To supply the evaporation loss, 0.0215 ft. in depth per day, from 164 sq. miles	1 140	sec-ft.
To supply leakage at structures	250	"
For power, and other uses	200	"
Total	1 590	sec-ft.

There is, of course, some uncertainty relating to the loss of water by evaporation. Experiments made at Boston, Mass., and at Kingsburg, Cal.,* indicate that, for a mean monthly temperature of 78.5° Fahr., which is about that which will prevail during the dry months at Gatun Lake, an evaporation of 0.0215 ft. per day may be expected from an open water surface. This is about 7% in excess of the evaporation rate assumed by General Abbot in 1905. Canal Commissioner H. H. Rousseau, Assoc. M. Am. Soc. C. E., in an address before the Trans-Mississippi Congress in August, 1909, places the evaporation loss at $\frac{1}{7}$ in. per day, but does not state how he arrives at this amount.

For a mean monthly temperature of 78.5°, the Boston curve indicates an evaporation rate of 0.021 ft. in 24 hours. The highest observed points on the Boston curve are for mean monthly temperatures of 72° and 70° in July and August, and corresponding relative humidities of about 71.5 and 75.5 per cent.

The Kingsburg curve indicates a rate of 0.022 ft. for a mean temperature of 78.5 degrees. The highest points on this curve, based on observation, are for mean monthly temperatures of 82° and 84°, in July and August, with corresponding relative humidities of 32 and 35 per cent.

On the Isthmus the mean monthly relative humidity during the dry months is about 84 per cent. It seems probable that the actual evaporation rate from a large body of water on the Isthmus, owing to the high moisture content of the atmosphere, will be somewhat less than the rate introduced into the calculation; but a satisfactory basis for a humidity correction is lacking.

* "Evaporation from the Salton Sea," by C. E. Grunsky, M. Am. Soc. C. E., *Engineering News*, Aug. 13th 1908, p. 163.

Mr. Grunsky.

The leakage at structures, above noted, is the estimate of the Board of Engineers of 1905. The requirement for power has also been introduced into the calculation at the Board's estimate. The present plans contemplate the use of much more water for power, but this is immaterial, as, in case of need, all available water would be used for lockages during the dry months and none at all for power, which can always be otherwise secured.

According to the above figures, on the assumption that the lake is full at the beginning of the dry months, there would be available for lockages and to supply the seepage and other indeterminate losses, 3 510 — 1 590 = 1 920 sec-ft. On the extreme assumption of no seepage loss and no allowance for evaporation from marginal wet lands, all of the 1 920 sec-ft. would be available for lockages.

Following General Abbot's method of estimating the quantity of water required per lockage, on the assumption that a reduced lock length will be used for 80% of the lockages, the average requirement per lockage per day through the Canal would be about 55 sec-ft. According to Mr. Sorzano, each lockage per day will require 96 sec-ft. It is probable, of course, that the Canal will be used for some time as assumed by General Abbot, but the possibility must also be admitted of a demand on the Canal to its capacity within comparatively few years. This capacity may be limited by the number of lockages that can be made per day, or by the number for which water is available. The maximum traffic, expressed in tonnage and not by the number of lockages, will undoubtedly be attained when the locks are always crowded full of shipping and no lockages are made with intermediate gates. By the method of estimating used by the Board of Consulting Engineers and by General Abbot, there should be required for each such lockage through the canal 6 900 000 cu. ft. of water, or 80 sec-ft. for 24 hours.

The available water, therefore, would permit of 35 lockages per day, if 80% are lockages with intermediate gates. On the author's assumption of the requirement per lockage through the canal, there would be water for 20 lockages per day. As the Board of Engineers would estimate (intermediate gates not used), there should be water enough for 24 lockages per day when the demands of commerce have reached the canal capacity. If no water were used for power, these figures might be increased by 4, 2.5, and 2, respectively.

The maximum traffic through the Canal, as limited by the speed with which vessels can be passed through the locks, is given at 40 per day by Alfred Noble, Past-President, Am. Soc. C. E., and Joseph Ripley,* M. Am. Soc. C. E. It is 48 according to Mr. Rousseau,† who probably voices the opinion of the Canal authorities. Messrs. Noble

* Report to the Board of Consulting Engineers of 1905.

† Address at Denver, in August, 1909, before the Trans-Mississippi Congress.

Mr. Grunsky

and Ripley place the extreme number at 57, but state that, after allowing for all causes of delay, it should not be hoped to realize more than 40.

These numbers are far in excess of the 20 to 24 lockages estimated from the water supply. The water supply, in other words, does not appear to be sufficient to bring the canal up to the tonnage capacity of the locks without utilizing more storage than is now contemplated. Each additional foot below the 82-ft. level drawn from Gatun Lake will add about 4.5 to 5.5 lockages per day during a period of 120 days, this number depending on whether the net contents of the locks, or the figures suggested by Mr. Sorzano, should be introduced into the calculation.

Any seepage losses from Gatun Lake will reduce the number of lockages by one per day for each 80 to 96 sec-ft. of such loss. That there will be some such loss seems unquestionable. If it should approximate the amount suggested by the author—about 1 960 sec-ft.—it would indeed be a serious matter. The writer sees no cause for alarm. He believes that this loss will be small, and that it should only be considered as one of the elements of risk which, in some measure, may reduce the efficiency of the lock canal. The distance which must be traversed by percolating waters from Gatun Lake to any outfall point, except near the various dams, is so great that the velocity of motion will be very slow. The measured losses of water from canals, as cited by Mr. Sorzano, can only be used as evidence that seepage losses are possible; they afford no criterion of the amount of such loss from a lake of large extent. The measurements in the case of the Kings River and Fresno Canal, referred to in the paper, were made by the speaker. The canal lies in a flat alluvial territory, and the seepage was but a few feet vertically downward to the water-table.

The available data indicate that the traffic capacity of the Panama Canal, during the dry period, will be limited by, and is to be determined from, the available water supply. The best method of holding the water supply throughout the year at the capacity of the locks is, therefore, a matter which should receive timely attention.

Mr. Hodges.

H. F. Hodges,* M. Am. Soc. C. E. (by letter).—Mr. Sorzano's paper on the probable water supply for the lock canal at Panama is most interesting. The conclusions he reaches are so different from those of others who have reasoned from the same data, that the writer feels impelled to join in the discussion which he so kindly invites.

The author's remarks on the requirements of navigation admit of no convincing contradiction. Anyone is at liberty to prophesy on this subject. The dimensions which are now fixed for the locks of the Panama Canal, and the depth which has been given to its channels,

* Lt.-Col., Corps of Engineers, U. S. Army.

express the judgment of the authorities responsible for these features as to the probable future needs of navigation. If the draft of vessels desiring to use the Canal should grow beyond its depth, however, it may be said that, not only the Canal, but the great majority of harbors now available, would have to be deepened in order to accommodate these monster vessels; and that the cost of deepening the Canal and renewing its locks would be only a small part of the expense to which mankind would be forced by such a development. Mr. Hodges.

The clearance which Mr. Sorzano regards as necessary (6 ft. below the keel in a reasonably good channel), seems a little large, and his requirement of 3 ft. below the keel in a lock is unquestionably more than is needed. Large vessels pass through the St. Marys Falls Canal drawing practically every inch of water which the elevation of the sill will permit, and sometimes, strangely enough, a little more.

Table 1 is possibly a little misleading in its reference to the actual and the assumed dry season. It is true that the actual number of rainless months on the Isthmus sometimes exceeds the ordinarily assumed length of the dry season, but as far as its effect on the water supply of Lake Gatun is concerned, this is not important. The rivers continue to run with much more than their ordinary dry-season flow for some time after the rains have ceased, so that if a rainless season of 5 months is assumed, there would only be a season of 4 months of minimum water supply in the lake. These periods, of course, are given in round figures, but they will justify Mr. Sorzano in assuming in his computations a dry season of only 120 days, although indicating one of more than 150 days in Table 1. Mr. Sorzano, however, has overstated his case in saying that "the average discharge of all the streams that can empty into the proposed high-level lake at Gatun, will rarely exceed 1 100 cu. ft. per sec. in dry years." The minimum daily average for the 4 months of the dry season, namely, the average in the worst known dry season for 19 years, is 1 190 cu. ft. per sec., and the average dry season has a daily average discharge of 3 030 cu. ft. per sec. For the 3 driest months, the discharge is less, but the quantity stored in the lake, if distributed over 3 months instead of 4, would more than make up the difference.

It is in its assumptions as to the consumption of water that the paper differs most from discussions which have led to the usual belief in the adequacy of the water supply. The assumption as to surface evaporation is somewhat greater than that which experience on the Isthmus seems to justify. The average evaporation for the dry months, shown by observations, is 6.6 in. per month for the 3 driest months, and 6.026 in. when the dry season is considered 4 months long. In the remaining months of the year it is approximately one-half of this. Mr. Sorzano's assumption of $\frac{1}{4}$ in. or more per day is somewhat in

Mr. Hodges. excess of the actual determined amount. The writer, however, believes that a credit to the water supply of considerable moment has been overlooked. The contribution to the waters of the lake during the dry season is the discharge of the Chagres River and its tributary streams. Its stated amount has been estimated, not from the rainfall and catchment area, but from the measured discharge of the rivers in the dry months. During these months, as well as during the rainy months, the area which will eventually lie under the water of the lake is now covered by a dense growth of lush vegetation, which, even in the dry season, draws up and gives off to the air a tremendous amount of moisture which never appears in the measured discharge of the rivers. It is known that the amount so given off by growing vegetation may reach approximately the figures assumed for the evaporation. This loss will cease when that from evaporation comes in. In debiting the lake with evaporation from its surface, one should not forget to credit it with the unknown amount by which the river flow is now reduced through the medium of vegetation growing over the lake bed. No fixed value has been attributed to this amount in any discussion of the lake supply. It has been held simply as an indefinite although considerable reserve. Mr. Sorzano's evident respect for border evaporation, or "fly-off," indicates that he will probably be sensible of the importance to be attached to this reserve.

In connection with border evaporation, Mr. Sorzano states that the drying up of the lake "in every dry season, will uncover an area of land measuring more than 100 sq. miles." This, of course, depends on how much of it dries up. The area of the lake, at + 87, is 167.4 sq. miles, and at + 80, is 153.3 sq. miles. The writer's studies lead him to believe that it will not dry up even as far as the latter reference, and if it does, Mr. Sorzano's area of 100 sq. miles will be reduced to less than 14 sq. miles. In the average dry season the writer believes that the lake will not shrink noticeably.

As for the percolation through the bottom and sides of the reservoir, the author admits that it can only be guessed at, and the writer gives cordial assent to that proposition. Efforts to determine it, however, have not been lacking. The gentleman whom Mr. Sorzano compliments as being a very able engineer, and from whom he quotes as to certain experiments for the foundations of the dam at Gatun, has also made quite extensive borings at different portions of the rim of the reservoir area and of its bed, in places where the configuration and geological formation of the soil indicated that such investigations were most necessary. The result of these investigations has not shown any area which can be regarded as dangerous, and has strengthened the confidence of the engineers on the Isthmus that the percolation through the bed and walls of the reservoir will not be a matter of serious importance.

In reference to this matter, the fact is worthy of notice that the Camacho Reservoir, to which Mr. Sorzano alludes as a horrible example of the permeability of Isthmian rock, supplies, from a water-shed of about 1 sq. mile, a total of about 2 000 000 gal. daily. This amounts to a flow of 3.1 cu. ft. per sec., aside from all losses due to evaporation, percolation, "fly-off," and what-not. Such a flow over the water-shed tributary to Lake Gatun, after deduction for the losses mentioned, would leave about 4 000 cu. ft. per sec. available, an amount greatly in excess of any probable need. Mr. Hodges.

The author's discussion of the consumption of water in lockage requires a little correction. Owing to the fact that the upper lock in the Gatun and Miraflores flights is, for certain reasons, somewhat larger in area than the lower ones, the lifts between the upper and lower pools at these localities are not divided equally among the locks, and, at Gatun, the normal lift of the upper 1 000-ft. lock will be 26.38 ft., when the lake is at + 85 and the sea below at mean level. The lift of the other two 1 000-ft. locks at Gatun will be 29.31 ft. each, at the same water stages. At Pedro Miguel the normal lift will be 30 ft. at lake levels of 85 and 55, and, at Miraflores, the normal lift of the upper 1 000-ft. lock, with the lake at + 55 and the sea at mean tide, will be 26.10 ft., while the lift of the lower lock will be 28.90 ft. These figures will alter somewhat the lockage prisms assumed by the author.

Neglecting losses by leakage, etc., the total draft on the summit level, due to the passage of a vessel through the Canal, should be two prisms of lift and no more. These two prisms are those in the upper locks at Gatun and at Miraflores, and since, for reasons which will appear later, the opportunity of saving water at Pedro Miguel is so much greater than at Miraflores, the latter place will rule the water consumption. The displacement of the vessel, although mentioned by Mr. Sorzano, does not have any influence on the net result of a complete passage. It increases the consumption for a vessel going up and decreases that for one on its downward lockage. Neither of the two prisms of lift which each passage takes from the lake, however, must, of necessity, be a full prism; and that leads to an investigation of the statement, inferentially accepted by Mr. Sorzano, that intermediate gates cannot be used to advantage in the three-lift flight at Gatun. This statement crept into the Report of the Board of Consulting Engineers of 1905, and, apparently, has received circulation and acceptance since. The writer craves indulgence for an endeavor to prove its fallacy.

In considering the operation of the twin flight of locks, it should be remembered that it is intended to reserve one flight for up-bound and one flight for down-bound vessels. The water in each lock of the up-bound flight will regularly be left at its high level, and in the down-

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bound flight, regularly at its low level. Let Fig. 1 represent a three-lift flight of locks, each divided, as are those proposed for Gatun, into two unequal parts by intermediate gates, *n-e*, *d-b*, and *l-m*. Let the chambers, *A*, A_1, and A_2, be equal in area, and call this area *A*, and represent the area of the lock, B_1, by *B*. Suppose a vessel to have passed down, using the 550-ft. locks; then, since it must pass from lock A_1 to A_2, through B_1, and the gate, *g-r*, must then be shut to lower the boat to sea level, it follows that the level in B_1 as well as in A_1, after the downward passage of the vessel, will be the high level of the lock A_2. Represent the lift between the high levels of successive subdivisions, as follows:

B to A = o;
A to A_1 = x;
A_1 to B_1 = y;
B_1 to A_2 = z;
A_2 to sea = w.

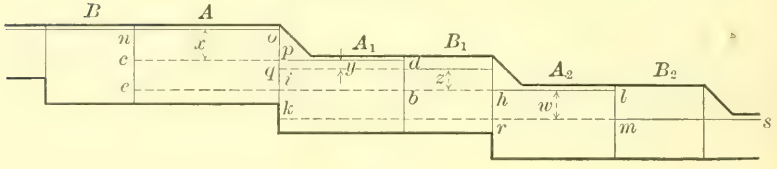

FIG. 1.

Fill *A* to lake level, *n-o*, and admit the vessel; close the barrier *n-e*. Then, since the level in A_1 and B_1 is the high level of A_2, represented by the line, *e-l*, the total quantity of water above that level, *e-l*, in the two locks, *A* and A_1, before equalizing, is represented by the prism, *n-e-i-o*. If, now, keeping the gates and valves in the barriers, *n-e* and *d-b*, closed, the valves in the barrier, *o-i*, be opened, the water level, *n-o*, in *A*, and, *i-b*, in A_1, will equalize at *p-d*. The prism, *c-e-b-d*, which is now above *e-l*, must be equal to the prism, *n-e-i-o*, which was above that level before equalizing. From this we have, since *o-p* is equal to *x*, *p-q* to *y*, and *q-i* to *z*, and the areas of *A* and A_1 are equal:

$$A\ (x + y + z) = 2\ A\ (y + z);\ \text{hence},\ x = y + z \ldots\ldots\ldots (1)$$

Since the barrier at *d-b* was kept closed, the level in B_1 during this operation has remained at *b-h*. Let the vessel now be passed into the lock, A_1, and the gate and valves in *o-k* be closed. In A_1 and B_1 the total prism above *b-h* is *p-i-b-d*, equal to $A\ (z + y)$. Open the valves in *d-b* and the water levels in A_1 and B_1 will equalize at some level, *q-g*, the total prism above *b-h* now being *q-i-h-g*, equal to $(A + B)\ z$. Equating these two prisms, we have:

$$A\ (y + z) = (A + B)\ z;\ \text{hence}\ z = \frac{A\ y}{B} \ldots\ldots\ldots (2)$$

Mr. Hodges.

Now open gate, *d-b*, and the valves in *g-r*, and equalize the levels between the combined chamber A_1-B_1, and the chamber A_2 in which the water level is at the line, *r-m*. Equalization will take place at some level, *h-l*, at a distance, *w*, above sea level, and, by the reasoning already followed, we have the prism, *q-k-r-g*, equal to the prism, *i-k-m-l*, or,

$$(A + B)(z + w) = (2A + B)w; \text{ hence } Aw = z(A + B) \ldots (3)$$

Combining Equations 2 and 3, we have $w = \frac{y}{B}(A + B) = \frac{A}{B}y + y$.

Combining Equations 1 and 2, we also have $x = \frac{A}{B}y + y$; hence $x = w$. We find, therefore, that *x*, which we know to be equal to $y + z$, is also equal to *w*, and consequently, that the lifts between consecutive 550-ft. locks are the same. Now admitting the vessel to the chamber A_2 and closing the barrier *g-r*, the water in A_2 can be lowered to sea level through the valves in *l-m*, and the vessel passed out through B_2 with the total loss of the prism, *h-r-m-l*, which is $A\ w$, and equal to the prism, $A\ x$, originally taken from the upper level. The levels in the flight will be left as found.

In order to accomplish the result it is necessary only that after the vessel has been admitted from the combined lock chamber, $A_1 + B_1$, to the 550-ft. lock chamber, A_2, closed below by the barrier *l-m*, the barrier *g-r* should be closed behind it before it is lowered from the level, *h-l*, to sea level. In this way the vessel drawing full draft can be passed through the flight with only the expenditure of water due to the prism of area *A*, or the 550-ft. lock, and the height, *x*, or $y + z$, or *w*, equal to one-third of *A*. The lock, B_1, becomes practically a portion of an intermediate canal, the water level never going below the normal high level in A_2, or *h-l*. There is no necessity for making two operations, as indicated above, of lowering the vessel from *p-d* to *h-l*. The valves in *d-b* and *g-r* may be opened at once, allowing the prisms in A_1 and B_1 to flow simultaneously into A_2, thus avoiding a slight loss of time.

If one takes the up-bound instead of the down-bound flight, the locks will regularly be found full, but the reversal of the operation will lift a vessel of suitable size from the low level to the high level with only the expenditure of one 550-ft. prism of lift, and without necessitating diminution in draft or delay in transit. A similar discussion will show that even the smaller 350-ft. lock may be used, with a very small expenditure of water, in locking small vessels through. The operation is somewhat more complicated, as the lock flight is divided into more lifts in order to accomplish this end, but the lifts are lessened and the time of each lockage, therefore, is made shorter; so that, while the lockages would be increased in number, it is doubtful if any increase in the time of passage would result. The saving of water through the use of the smaller locks is continuous as long as the series

Mr. Hodges. of small lockages remains uninterrupted. Under certain circumstances, the interruption of the series by a vessel requiring the full size and full depth nullifies the saving due to one of the previous lockages.

The writer feels that he owes an apology for the length of the foregoing demonstration, but he has thought it necessary in view of the general acceptance of the statement that intermediate gates cannot be used to advantage in the three-lift flight.

There is also another means of saving lockage water, of which Mr. Sorzano had probably no information. The middle walls of the locks are designed so that communication, controlled by valves, can be opened between the twin-lock chambers, and one can be used as a reservoir for a portion of the prism of lift in the other. A lockage in one direction may then require only a portion of the normal prism, leaving the remainder stored up for use in the upper twin lock of the other flight. Such saving in case of the flights is possible only with vessels which do not require the full draft of the normal prism over the sills, but it is very evident that the great majority of vessels will be of this description, especially those which can pass through the smaller locks. The possible saving due to this cross-storage of water is greatest at Pedro Miguel, where there will be no loss of depth by cross-filling, and where, under the most favorable circumstances, 50% of the lockage water can be saved, and under ordinary circumstances a quantity varying between 25 and 50 per cent. It is less at Gatun where the maximum possible saving is about 24%; and it seems safe to count on about 15 per cent. At Miraflores, the saving should be intermediate between that at Gatun and at Pedro Miguel. The existence of this possible saving is the ground for the previous statement that the expenditure of water will be ruled by Gatun and Miraflores, rather than by Gatun and Pedro Miguel.

Mr. Sorzano may be correct in his opinion that the development of commerce will lead to the use of the large locks with a number of vessels, instead of the small locks with one or two ships. If this be the case, each lockage will mean several passages instead of one or two, and the net result to commerce will be approximately the same. Each one of the fleet lockages will take a longer time than the one involving a single vessel, and, consequently, there will be fewer of them in the course of the 24 hours.

It is difficult to foretell just what line of development commerce will take. When the Poe Lock at St. Marys Falls Canal was designed, its dimensions were based on the methods of Lake carriage then existing, with one steam barge and a number of tow barges, it being intended to take in, as far as possible, one entire tow, two ships abreast. Methods of carriage on the Upper Lakes have changed since then, the large single vessel having come to the front; and if the lock were to be

built now, less attention would be given to taking in a fleet of relatively small vessels two abreast, and more to taking in two large vessels tandem. The canal on the American side at "The Soo" now averages approximately one and one-half boats per lockage. Mr. Hodges.

The author's intimation that the use of the intermediate gates will be objectionable on account of the great number of safety appliances to be used for each vessel at each lock, and the necessity of moving all vessels at extremely slow speed, can apply only to the use of the 350-ft. lock. No more movements are necessary in locking with the 550-ft. lock than with the 1 000-ft. lock. As stated before, however, when the small 350-ft. lock is in action, the method of operation may be such as to increase the number of lifts in order to save the greatest possible quantity of water. Nevertheless, it is not necessary that the number of lifts should be increased, even with this small lock, unless full draft is needed, and, even should it be thus increased, the lifts themselves are correspondingly diminished, the time of each lockage is correspondingly reduced, and, furthermore, the small size of the vessel will render its movement from lock to lock more easy and rapid. The writer is not inclined to accept, without reserve, the general statement that the use of the intermediate gates will be attended by delay, believing that the reverse will be the case.

The author's estimate of the water used for hydro-electric plants indicates that he has been misled by the knowledge that the Commission is now building two generating plants at Gatun and Miraflores. Apparently, he has assumed that it is intended to generate the full power at these two localities, and, therefore, he has stated that "it is intended to use about 6 000 h.p."

The present intention is to develop for operation somewhat less than 2 000 h.p., and this at Gatun. Ordinarily, the Miraflores plant is intended to lie idle, and only a part of the Gatun plant is to be in use. During the period of construction it is necessary to have the two plants, and, using steam, to develop a power considerably greater than is contemplated for the operation of the Canal in after years, when the power will be developed by water. The calculations which have been made on the Isthmus as to the water supply assume an expenditure of 275 cu. ft. per sec. for the permanent development. After the Gatun plant begins to supply the operating needs, the Miraflores plant, which will still have auxiliary boiler capacity, will be held in reserve ready to be thrown into the line in case of any failure. Having boilers installed, steam can be used should it be found desirable for any reason, in which case the estimated expenditure of 275 cu. ft. per sec. from the lake will cease.

In the author's discussion of the losses due to gates and valves, he has been rather hard on the Canal. It is true that the Gatun locks

Mr. Hodges. will have 40 gate leaves and a great number of valves, but there are only 4 gate leaves and 3 sets of culvert valves there through which the lake will lose water. The others will have their own leakage supplied by the leakage from above. If it be assumed that there is an opening $\frac{1}{32}$ in. wide around the meeting faces of the effective gates and valves, the leakage at Gatun will not reach half the amount mentioned by Mr. Sorzano. The assumptions made on the Isthmus as to the leakage through gates and valves, including the spillway, give a combined total of 275 cu. ft. per sec. for both ends of the upper level.

Any conclusions as to the water to be wasted in maintenance operations (Cause VII in the paper), must be largely guesswork. Undoubtedly, however, most of the intentional operations which will involve waste of water can be made to occur during the rainy season, when the loss is of no consequence. The writer cannot possibly accept the author's claim that 125 cu. ft. per sec. must be chargeable to these operations, including the power plants and hydraulic motors operating on Lake Gatun. If these plants and motors are private property, it will be a very simple matter for the Government to stop any draft of water should it be found desirable. It is not part of the duty of the Canal to supply them. If, on the other hand, the author refers to the Government plant, that has been reckoned in elsewhere.

The author's statement that the safety appliances and towing devices, now arranged for double-track service, will make single-track operation very dangerous, does not appear to the writer to be justified. Single-track operation of a flight of locks is undoubtedly less convenient than double-track operation, but the safety appliances and towing devices, to which the author alludes, introduce into the single-track operation no element of danger not present in the double-track operation. The twin flights of locks are intended to be entirely independent of each other in this regard.

The author's Cause IX, "Losses due to False Maneuvers and Accidents," is also a matter of speculation. He mentions, however, the 1 104 hours of stoppage due to breakage at the St. Marys Falls Canal, Mich., during its 2 007 days of operation, as an indication of the amount of water loss which may be expected in the Panama Canal. There is no statement of how much water loss at "The Soo" was involved by these hours of delay. From the best of the writer's information, he is inclined to believe that there was considerable time lost, but very little water.

The losses at the spillways and regulating gates of the Panama Canal can be controlled to a certain extent. The discharge over the spillway is to be regulated by crest gates about 19 ft. high, under the lower edge of which the flow will take place. When these gates are closed, it does not seem to be impracticable to pack the vertical edges

and sill in such way that there will be little, if any, leakage, even Mr. Hodges.
under the 16-ft. head to which they will be exposed.

The author's Cause X, "Miscellaneous Losses," also seems to the writer to be somewhat overdrawn. Among these losses he includes all the needs of the population and industries which may locate around the borders of the lake and on its numerous islands. The population of the country during the construction of the Canal is very much greater than it was before, and the needs of this increased population are now adequately supplied by reservoirs which lie either above the level of Lake Gatun or are not tributary to it at all, and the waters impounded in these reservoirs are not counted in the estimated lake supply. If these reservoirs prove insufficient for future consumption, they can be supplemented by others which will store wet-season water for dry-season use, without affecting the dry-season supply for Lake Gatun. Mr. Sorzano also mentions the consumption and replenishing of all passing vessels and craft using the Canal. If this prove a source of inconvenience, the vessels can be made to get their water supply elsewhere, as they do now, and as they would have to, were the Canal to be built at sea level.

He also mentions the losses and waste at the intermediate section from Pedro Miguel to the sea-level section at Miraflores, due to evaporation, percolation, "fly-off," etc. Of course, there will be evaporation from the small lake at Miraflores, but it also has certain small independent sources of supply which are greater in proportion to its area than those of Lake Gatun. It need not draw on the latter to supply its losses by evaporation, etc.

Assuming the area of Lake Gatun at 164 sq. miles, and the depth of storage available, as stated by Mr. Sorzano, at 7 ft., namely, from + 80 to + 87, the total volume of water thus stored is equivalent to a flow of 3 090 cu. ft. per sec., for 120 days. Adding this to the minimum supply of 1 190 cu. ft. per sec. gives a total supply of 4 280 cu. ft. per sec. available for all uses during the dry season of 4 months, without drawing the lake down below + 80.

The observed rate of evaporation during a dry season of 4 months is equivalent to a flow of 930 cu. ft. per sec. for the lake area. In his calculations, the writer has assumed the leakage at the gates and valves at 275 cu. ft. per sec.; seepage, waste, etc., at 85 cu. ft. per sec.; and lighting, power, etc., at 275 cu. ft. per sec. In this assumption, as well as in Mr. Sorzano's, the credit due to saving in the lake the quantity of water which is now poured out by the vegetation covering its bed has been neglected, and the element of percolation in the bed, which is entirely unknown, has been omitted. The quantity of water required for lockage has been obtained by the following assumptions.

Mr. Hodges. which may or may not prove to be justified in fact. The writer gives them for what they are worth.

TABLE 8.

(1) Lock.	(2) Area.	(3) Normal Lift. Gatun.	(3) Normal Lift. Miraflores.*	(4) Total water for complete passage, in cubic feet.	(5) Column 4: in cubic feet per second.	(6) Weight.	(7) Column 5 weighted.	(8) Saving due to cross-filling.	(9) Column 7 corrected for Column 8.
1 000 ft.....	133 800	26.38	26 10	7 022 000	81.5	5%	4.08	16%	3.42
900 ft.....	122 800	28.10	27.40	6 810 000	78.9	15%	11.83	16%	9.93
550 ft.....	82 400	28.33	32.75	5 033 000	58.3	50%	29.15	15%	24.77
350 ft.....	40 400	18.58	30.16	1 969 000	22.8	30%	6.84	11%	6.08
Totals.....	.....	.....	.....	.....	.....	.....	51.90	.....	44.2

* No intermediate gates in lower lock at Miraflores.

The plan of the upper lock is such that lengths of 1 000 ft., 900 ft., 550 ft., or 350 ft. can be used, the 550-ft. and 350-ft. lengths being capable, by squeezing, of increase to about 600 ft. and 400 ft., respectively. In preparing Table 8, the assumption has been made that 5% of the vessels arriving would require the 1 000-ft. lock; that 15% would require the 900-ft. lock; that 50% will pass in the 550-ft. lock, and the remaining 30% in the 350-ft. lock. With these assumptions, and taking into consideration the saving in water which can be effected by cross-filling, the total average consumption for one passage per day, *i. e.*, for two prisms of lift each day, is 44.2 cu. ft. per sec. Neglecting any advantage to be derived from cross-filling, it is 51.9 cu. ft. per sec. Deducting the sources of loss assumed above from the total available supply would give 4 280 — 1 535 = 2 745 cu. ft. per sec., which, at the calculated value for cross-filled lockage, will allow 48 complete passages of the Isthmus per day with a remainder of 623 cu. ft. per sec. as a margin. The writer believes this margin to be sufficient to cover underestimate of losses, and feels confident that the water supply will prove adequate to the needs of the Canal for a great many years, if not for all time to come. Should it ever become necessary to increase it, this can be done very simply by the construction of a dam on the Upper Chagres River, at Alhajuela or elsewhere, which will permit of increased storage to supply the deficiency during the dry season. The duty of building this dam may be properly left to posterity, should the need ever arise.

The foregoing estimate of the water supply is based on the minimum observed daily average for the 4 dry months, *viz.*, 1 190 cu. ft. per sec. The daily average supply for the average dry season of 4 months is much greater, being 3 030 cu. ft. per sec. In the average year, therefore, the surplus will be greatly increased.

In the foregoing, as in Mr. Sorzano's paper, a 7-ft. storage, from + 80 to + 87 in Lake Gatun, has been assumed. In the recent plans, the

storage has usually been taken at only 5 ft., from + 82 to + 87, and even with this reduced variation in level, it is thought that the supply is sufficient for all lockages through a dry season of minimum flow. In other words, instead of the lake level being reduced, as concluded by Mr. Sorzano, it will not be necessary to bring it below + 82, giving a depth of 42 ft. in the Culebra Cut, under the most unfavorable circumstances. For many years it may be held desirable to operate at a lower level, until vessels requiring the full draft desire to use this route; but, in the writer's judgment, there will be no necessity, due to scarcity of water, for such low-level operation. Mr. Hodges.

THEODORE PASCHKE, M. AM. SOC. C. E. (by letter).—The author has treated his subject most ably. His conclusions are startling, and if, after a full discussion, they still prove to be approximately correct, they are a severe arraignment of the designs for this great work. Mr. Paschke.

Of the various items affecting the water supply of the Canal, Cause III, "Percolation through the bottom and sides of the reservoir," has attracted the writer's particular attention, as it embraces a feature which has not been mentioned in the paper, and, apparently, has been overlooked by Mr. Sorzano.

The author's deduction that the water level of the lake will suffer a certain lowering during the 4 months of dry season, by percolation or seepage through the bottom and sides, is based on certain assumptions of the existence of fissures and seams in the rock formations and the general porosity of the subsoils in the locality, all of which in turn have been the subject of close scrutiny, investigation, tests, and other researches by the Canal engineers. These assumptions, therefore, are fair and are not to be gainsaid.

For the sake of argument, it is also admitted that the deductions therefrom—the amount of seepage—will conform to the conditions existing when the reservoir is first being filled. From that time, sedimentation will take place all over the bottom and sides of the lake, and attention is called to this feature. If the writer is not misinformed, the soils which overlie the rock formation of the Isthmus are composed mainly of volcanic deposits. These soils yield easily to the erosive action of running water resulting from the torrential downpours peculiar to the rainy season in the tropics.

In a stream like the Chagres, having its origin in a mountainous region overlaid by such volcanic deposits, the flood waters are charged to their capacity with the eroded particles of the soil. The finer particles, held in suspension, are never deposited along the sides or bottom of the stream, but, impelled by the current, are carried out to sea to furnish material for new combinations and formations in Nature's great workshop. In cases where the flood waters overflow the banks of the stream into adjoining and comparatively level regions, where the flow of the waters is arrested, the suspended matter is slowly

Mr. Paschke. deposited, thus forming, in the course of time, a well-nigh impervious layer over these stretches, constituting the basis for the swampy character of regions which otherwise, due to their elevations, should drain themselves.

Only the coarser and washed particles carried by the flood waters are found deposited in old and new river beds. The opinion is here ventured that the low hills of the Gatun region consist of such washed-out material deposited from time to time by the flood waters of the Chagres. This would explain the high seepage flow through some of the Gatun soils given in Table 3.

By throwing the artificial barrier across the Chagres Valley at Gatun, the waters of that river will be impounded and form a lake. The effect will be that all the sediment brought down by the flood waters during the rainy season will be deposited in this lake. The coarser and heavier particles will be deposited at the point where the flood waters come in contact with the lake water, and the lighter and finer particles will be carried a little farther before they settle to the bottom. The lightest and finest particles, however (the volcanic dust particles in combination with organic matter held in suspension), will be diffused throughout the waters of the lake, reaching every nook and corner, where they will gradually settle to the bottom. This process of settlement of the finer particles will go on with more or less uniformity throughout the rainy season, which lasts about 8 months of the year.

It is not difficult to foresee the final result, provided the Gatun Dam stands and does not slip and disintegrate (as predicted by some), or is not washed out on account of some ramming accident at the locks (as predicted by others).

Granted that the soils forming the sides and bottom of the proposed lake are porous and that the underlying rocks are shattered, full of seams and joints, and that for these reasons the lake bed, at the beginning, will be nothing more than an immense filtering bed, surely the sedimentary deposits from the Chagres and other inflowing streams will eventually choke up the filtering features and ultimately seal effectively every pore, seam, and joint in the material forming the bed. Indeed this whole process is nothing more than a gradual filling up of the lake with the spreading of a water-proof blanket over the bottom and sides in advance of the coarser filling material.

The views here advanced are not born of a desire merely to criticize, but represent the conclusions reached during a professional experience of 12 years in one of the Central American States, during which the writer had ample opportunity for observing similar phenomena in reservoirs, lakes, and rivers.

In conclusion it should be stated that these views should not be taken as a defense of the present designs of the great work. Any possible modification of the author's calculations, due to any tendency to retard and check the percolation of water from the lake, will be so

slight as not to affect in the main his general conclusions, which, in the writer's opinion, stand unimpaired. They are an appeal not only to engineers, but also to the whole American people, to pause and think a moment over the possibility of having the traditional national reputation for sagacity come to naught by any short-sightedness displayed in the persistence of the present designs of a lock canal. Mr. Paschke.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—The writer will discuss only one feature of this paper, namely, the loss of water by percolation. The author's assumptions as to percolation differ radically from those which the writer has made, and the difference represents the greatest single difference between his calculations and the hydraulic calculations which were used in making the Report of the Consulting Engineers in February, 1909, to which the author refers. Mr. Hazen.

Under the heading "Seepage Velocity in Gatun Soils," reference is made to certain experiments by C. M. Saville, M. Am. Soc. C. E., Assistant Engineer of the Commission. From this report the author has derived conclusions which differ from the writer's, and it seems best to take up these experiments in somewhat greater detail. The experiments on soils are described on page 138, *et seq.*, of the Report of the Isthmian Canal Commission for 1908.

The soils were placed in galvanized-iron tanks, 39.36 in. in diameter, and 54.5 in. high. The tanks were coated inside with pitch and coarse sand. In the center of the bottom a connection was made which was protected by a piece of wire screening. Two $\frac{3}{4}$-in. pipes, with perforated cross-pieces, were carried through the sides of the tanks and embedded in the material to be tested. These were connected on the outside with pressure gauges in which the differences in head were observed. The water passing was collected and measured. As far as possible, an effort was made to deposit the material in the tanks in a condition similar to that when found in place. Every effort was made to drive out all the air from the material, and a constant head of water was maintained.

The discharges under conditions when the hydraulic slope equaled 1, or, in other words, when the head available to make water pass through was equal to the depth of the material, for the five materials tested, were as follows:

	Cubic feet per acre per 24 hours.
Stiff blue clay from test pit down the Chagres River, north of the dam site	49.9
Red sandy soil from embankment slope of spillway hill	226.7
Medium fine sand from test pits down the Chagres River	206.7
Brown sandy clay from Gatun Island test pit, between surface and 13 ft. below	136.4
Blue sandy clay from Gatun Island test pit, from 15 to 37 ft. below surface	295.7

Mr. Hazen. Mr. Saville did not reduce these to inches per day, but the writer has done this, with the following results, for the five materials, respectively:

0.014,
0.062,
0.056,
0.037,
0.081,

average, 0.050, equal to $\frac{1}{20}$ in. per day.

In applying these results to the probable percolation of water from Gatun Lake, allowance must be made for the probable difference in hydraulic slope. Certainly the average distance from the water in the lake through the ground to a point where it could discharge outside would be a considerable number of miles. For the purpose of this calculation, assume that it is only 1 mile, and that the discharge in all cases is at sea level. There will then be a head of 85 ft. acting in a distance of 1 mile, or a slope of 1.62% in place of 100 per cent. By a well-known law of percolation, the rate of seepage will be reduced in the same proportion as the slope, and the percolation to be expected will be $0.05 \times 0.0162 = 0.0008$ in. per day.

This rate of seepage is not to be reckoned necessarily on the whole area of the bottom of the lake. It applies instead to the smallest section of the pervious material through which the water is obliged to pass on its way from the lake to the sea. There is no means of determining what this smallest section would be; certainly it would be only a fraction of the actual area of the lake. It is needless to attempt to determine it, because even if the rate be applied to the whole area of the lake, it amounts to less than $\frac{1}{1000}$ in. per day, or to the one six-hundredth part of the amount assumed by the author.

Mr. Saville's experiments with rocks, made by cementing a piece of rock into a 6-in. pipe, represent more difficulty and perhaps less conclusive investigations. The results, for 1 ft. in thickness of stones, scaled from Plate 103, accompanying Mr. Saville's report, are given in Table 9.

Although these figures exceed considerably those found for the soils, no effort will be made to carry the calculation further, as it is obvious that the rates of seepage indicated are too small to be of practical significance.

The materials on the Isthmus, as the writer saw them in February, 1909, are generally of a ferruginous, clayey nature. That is, they are clays and shales, part of the alumina in which has been replaced with iron to the extent that they contain something like 10% of metallic iron. The substitution of iron for alumina changes the physical properties of the materials to such an extent that Northern experience cannot be applied safely, and their ability to stand up in stable embank-

ments must be based on actual experience with them rather than on Mr. Hazen. considerations of Northern precedent.

TABLE 9.

No.	Kind of stone.	Source.	Head, in feet.	Percolation, in cubic feet per square foot per hour.	Corrected Percolation, in Inches per 24 Hours	
					Head equals slope.	5 ft. head in a distance of 1 mile.
1	Blue sandstone	Lock site..	4	2.0	144	2.3
2	Volcanic ash	Lock site..	40	2.2	16	0.27
3	Volcanic tufa	Quarry....	40	0	0	0
4	Dark conglomerate	Lock site..	40	0.01	0.7	0.011
5	Brown conglomerate	Lock site..	40	0.12	0.86	0.014
6	Volcanic ash	Lock site..	40	0.21	1.50	0.024
7	Light blue-gray conglomerate	Lock site..	40	0.28	2.0	0.032
8	Argillaceous sandstone	Spillway...	40	0.40	2.9	0.047
9	Gray sandstone	Spillway...	40	0.26	1.9	0.030
10	Light brown sandstone	Spillway...	40	0.71	5.1	0.082

In the whole line of the work traversed by the Canal there is no considerable deposit of silicious sand or gravel. The so-called sandstone does not contain silicious sand particles, but, in appearance and in physical properties, is more like shale than the rock which we know as sandstone. This so-called sandstone, said to be water-bearing, does not permit percolation through its pores. There are occasional seams in it which carry limited quantities of water, in the same way that shales and mica schists and granites are fissured, and carry water in limited quantities.

All these materials, as seen by the writer in the various cuts on the work, and as tested by Mr. Saville in his laboratory, are substantially impervious.

Of course, it is true that the engineers did not have an opportunity to see every part of the lake bottom. That would have been impossible under existing conditions, and especially in a limited time, but the materials which they did see extending from one end of the lake to the other, and, in many places to its sides, were all of the same general class, and were all generally impervious, and there is every reason to believe that the materials throughout the whole lake are of the same general character.

There is no more reason to apprehend loss of water from Gatun Lake by percolation than there is to expect loss in the Ashokan Reservoir now building by the City of New York. No allowance for such percolation has been made for the Ashokan Reservoir, as experience with the Croton Reservoir, the Wachusett Reservoir, and a hundred smaller reservoirs, has not indicated the necessity for it.

Mr. Hazen. The author speaks of some of the dams built for water-works' purposes on the Isthmus, and of the leakage from them. The writer visited several and noted the conditions of leakage. These dams were made by filling with the materials most conveniently available to a not very large earth dam section, and apparently without any very deep preparatory excavation of the underlying material. In all cases there was a certain amount of seepage at the bases of the dams, coming apparently through the material under the dam a short distance below the natural surface. In one case a slight indication of leakage through the lower part of the embankment was observed. In no case was there indication of leakage through the deeper underlying materials.

The writer is at a loss to account for this percolation through the material a short distance below the natural surface. His first thought was that it might be due to the spaces left by decayed roots, but observation in cuts showed that as a rule roots did not extend far into the soil, and the seepage was through a layer deeper than that to which roots ordinarily extend. The writer has been told that in very dry seasons there are shrinkage cracks, due to the continued extraction of moisture, by roots and otherwise by evaporation, from the clayey soil containing a large percentage of pores filled by moisture, and that these shrinkage cracks sometimes extend to considerable depths. It may be that these cracks when once formed do not completely heal in the ensuing rainy season, and that there may be seepage through them.

The seepage from these reservoirs used for water-supply purposes in no case seems to be enough to affect the stability of the dams, nor to take away to any considerable extent from the quantity of water available for water-supply purposes.

The writer is interested to note that Mr. Sorzano raises no question as to the loss of water by seepage through the Gatun Dam itself, and he believes that all parts of Gatun Lake formed by it will be equally tight.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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RIVER PROTECTION WORK ON THE KANSAS CITY SOUTHERN RAILWAY, NEAR BRADEN, OKLA.

Discussion.*

By Charles H. Miller, M. Am. Soc. C. E.

Mr. Miller. Charles H. Miller, M. Am. Soc. C. E. (by letter).—Having visited this work several times, and having become familiar, to some extent, with the David Neale system during the past three or four years, the writer has read with much interest Mr. Lahmer's detailed description.

When the writer began the investigation of this system several years ago, he was confronted with the facts that there were no accurate cost or location records covering the work which had been placed, some of it, as many as twelve years before; that often there was no way of telling whether or not the changes occurring in the vicinity were brought about by the influence of the dikes or through natural causes; that some of the dikes seemed to have served well the purpose for which they were constructed, while others were found to have been entirely flanked and dropped into the bed of the river; that is, the river got behind the dike and continued to scour away the bank, until, in some cases, the dike was left on the opposite or bar side of the river.

In most cases it developed that no repairs or additions were made after the work was first constructed, it being the contention of the inventor that after it once became covered with sediment, it would be practically permanent.

* This discussion of the paper by J. A. Lahmer, M. Am. Soc. C. E., printed in *Proceedings* for October, 1909, and presented at the meeting of November 3d, 1909, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Miller. One point, however, seemed to come out prominently in most cases, namely, that the dikes possessed an inherent strength against being broken up piece by piece and carried off by the currents, and although entirely cut down by scour underneath and behind, and much bent and twisted, the main mass, especially that under the low-water level, remained, those parts above low water sustaining losses of one or more layers due to the decay of some of the materials composing them.

With this quality in view, this system was adopted, under the assumption that sufficient repairs could be made from time to time to hold the dikes in effective position, and, in the end, that a protection would be obtained at less cost than for a continuous system.

At Myrick, Mo., on the Missouri River, dikes have been in place for three years, and their position is being maintained, but this is much too short a period in which to obtain a reliable comparison between this and other systems. It has developed that an error was made in placing these dikes too far apart, and it has been necessary to build short pieces of bank revetment, both above and below them, for distances of approximately 100 ft., in order to prevent them from being cut down. The original cost of this work was $2.50 per lin ft. of bank; the repairs and additions placed amount to $1 per lin. ft., making a total of $3.50 per lin. ft.

This system does not contemplate any particular size or shape of dike, but fits the physical conditions as well as the class of materials to be found in the locality, and, naturally, there is a difference of opinion as to the most effective shape. The writer now uses a form which differs very much from that described by Mr. Lahmer. It is triangular or pyramidal in shape,* and the main part does not jut out into the stream as much as the long and narrow dike described in the paper. This triangular dike is now modified by discarding the large fascines for the foundation, and simply using a mattress, either of the solid or of the mud-cell type; it having been demonstrated clearly that the fascine method is the more expensive, first as to its actual construction, and second, because it permits more scour under the edges, hence more settlement, and, consequently, more material is needed to construct the dike to the required height. The mattress foundation is constructed so as to extend out beyond the edges of the proposed dike, and, as the scour progresses, this extended portion of the mattress, being pliable, drops and prevents excessive underscour. The mattress, extending a short distance above and below the dike, also prevents the eddy which cannot be entirely overcome, from cutting behind the upper bank part of the dike.

In calculating the relative effectiveness of the different forms of dikes much depends on the height and duration of floods, but the writer is firmly convinced that for the Missouri River, where floods often

* Described in detail and illustrated in *Engineering News*, October 22d, 1908.

remain at bank-full stage for a month or two at a time, the form of dike described by Mr. Lahmer will not hold up as well as it will in the Arkansas River, where the floods are usually of much shorter duration and that it will be necessary to place some top-bank protection both above and below the dike. In other words, the extent of the dike, up and down stream, at the top of the bank, will prove to be insufficient for a long period of high water. The dike described is intended to throw the main body of the river so far away from the main bank that the influence of any eddies which may develop, will not be felt at the bank. This will be found to be true for all ordinary stages of the river, at which times a considerable portion of the dike is above the water surface, and the water is forced around it at some distance from the general shore line or edge of the water. This distance gradually diminishes as the river rises, and when it is high, the water flows over the dike only a few feet out from the general shore line, and there is then nothing to prevent it from scouring out under the edge of the dike. Mr. Miller.

It is true that the stronger force of the currents is farther out and lower down from the top bank; but there is often sufficient force along the top bank during high water to cause much damage to such work. Mr. Lahmer calls attention to a caving, from 8 to 10 ft. in width, below Dike 5. It is just these small amounts at each recurrent flood which will soon result in the dike being on a narrow strip jutting out from the main bank, and this strip is usually quite susceptible to being cut out by eddy currents, thus allowing the upper part of the dike to settle down, thereby giving the currents freer play and a chance to scour more rapidly.

The necessity for extending the system of dikes or protection work such a great distance up the "bend" is not quite clear, in fact, it is believed that it would have been best not to have done so, at least during the first year. The best information at hand discloses no caving in this part of the river of an amount greater than about 300 ft. per annum; it is usually very much less; and, therefore, it is safe to predict that the railway tracks would have been perfectly safe for at least one year more, if no work had been placed above Dike 2, which is about 500 ft. from the track. Then, after the caving had continued so as to form a "point" in the "bend" at Dike 2, one or two additional dikes could be built each year, as the scouring progressed up stream, the net result being that new work would occupy the position of greatest attack as it moved up stream, and that the "false point" would tend to deflect the main river below the same so as to relieve the lower and older dikes. In fact, it is highly probable that it would deflect it so as to cause the sand bar to scour off to such an extent that the river would give no further trouble below Dike 8, and that by the time Dikes 2 to 8 became dilapidated from age they would no longer be needed.

Mr. Miller. The form of the "bend," as now protected, is that which usually obtains in this part of the river, and it is quite likely to remain so for many years; which means that all these dikes must be maintained continuously, also that the tendency of the river will be to cave in gradually and straighten out below Dike 8, thus necessitating additional work to protect that part of the railway track.

If the plan to stop the upper end of the work with Dike 2 had been adopted, this dike, being in a very exposed position, would no doubt have been damaged considerably and possibly cut down entirely; however, it would have been much reinforced on account of its exposed position, and in all probability would not have been entirely lost. In any event, the track could not possibly have been reached, the desired "point" would have been formed, and the conditions for this particular case much improved. It is the almost positive assurance of a loss of some extent, when the head of the protection work is left within the limits of active caving, which is objected to so strongly by many, and is the reason for extending the work to a point above the caving, as was done in the work at Braden.

It is hardly safe to depend on the erosion noted on the north bank of the upper end of the large bar at the point marked "vertical bank" on Fig. 1, nor to assume that eventually it will extend down so as to bring the river below the dikes, thus obviating the necessity of maintaining them, because this would be a very unusual manner for this caving to progress. Had Fig. 1 been extended to show the "bend" above this point, it is quite likely that it would have disclosed a greater rate of erosion or caving at some distance above the "vertical bank," tending gradually to throw the impact of the river away from the "vertical bank," and also tending to keep the river directed toward the upper dikes in the protection system.

On the other hand, if this unusual state or condition did occur, it would be one under which the dike system would be extremely hard to maintain, the one under which the continuous system of revetment or protection would be the most economical in the first place, because it would be necessary no doubt to place continuous protection between the dikes as they came within the line of almost perpendicular attack.

The dike system does not give the same security, in fact, as the continuous system, and where the bank is as near the railway as is the case in the vicinity of Dikes 6 and 7, careful and constant watching will always be necessary during each flood period.

With the entire "bend" protected as it is now, no marked relief can be expected. In this case, however, the top bank is composed in large part of a hard clay, and, therefore, the progress of any caving which may develop when the river is high will necessarily be quite slow.

One reason for using the dike system is its economy as compared with the continuous system, brought about chiefly because the dikes

deflect the currents so that part of the bank needs no protection; and much greater economies can be brought about when advantage can be taken of a "point" in a "bend" so that it can be held as an enormous dike. Mr. Miller.

Mr. Lahmer describes the use of large bundles of brush or hollow fascines as temporary protection against the rapid erosion just before the general work was started, and again to stop the caving just below Dike 5 during the flood which took place after its completion. It is for such emergency cases that these large fascines are especially useful, the writer having on several occasions successfully stopped very rapid caving with them. These fascines can be built on the river bank, and launched into place without any floating or special plant of any kind.

Any variety of brush and poles can be used, and the size may be quite variable; if the brush contains no leaves and small branches, then weeds, hay, or straw must be used to form tight sides and partitions. It is also possible to use sawed lumber instead of brush and poles. Telegraph wire may be used to bind the bundles or fascines together, and several wires can be twisted together to form cables or strands for anchoring the fascines to the bank. Some spikes can be used to advantage also. In cases where the river does not carry much sediment, and, in any event, it is well to have some empty sacks to be filled with soil and placed within the fascines to cause them to sink into place more quickly.

All or a sufficient quantity of these materials can usually be obtained readily and rushed to the point of danger by a work train. Large gangs of laborers can build one or more fascines simultaneously, and roll them into the river as fast as they are constructed. The stage of the river is immaterial, as long as the area of the bank or dry land on which the fascines are to be built, is large enough.

It is earnestly hoped that Mr. Lahmer will present figures showing the cost of maintenance.

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